HANDOUT

Tunnelling

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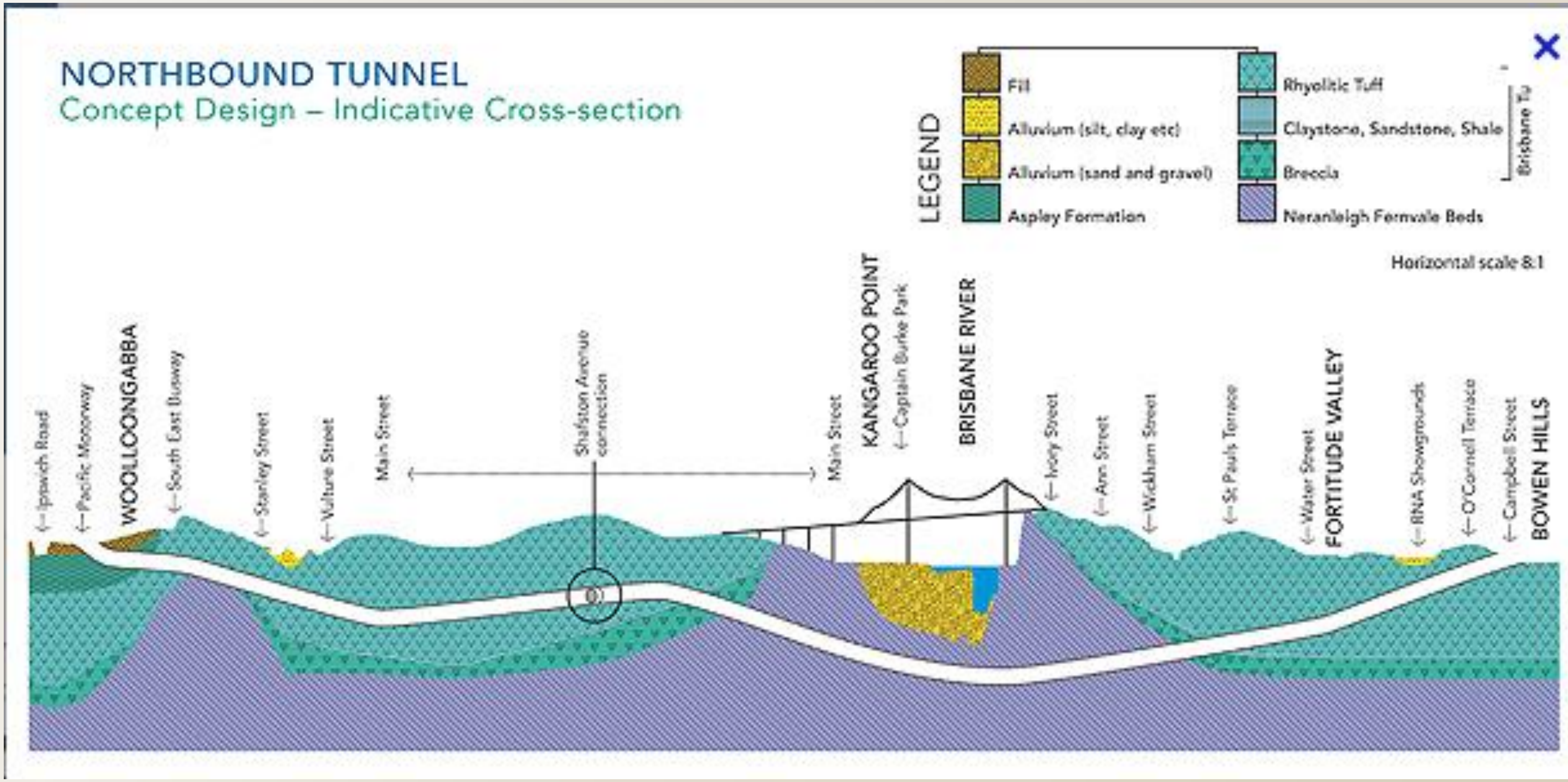
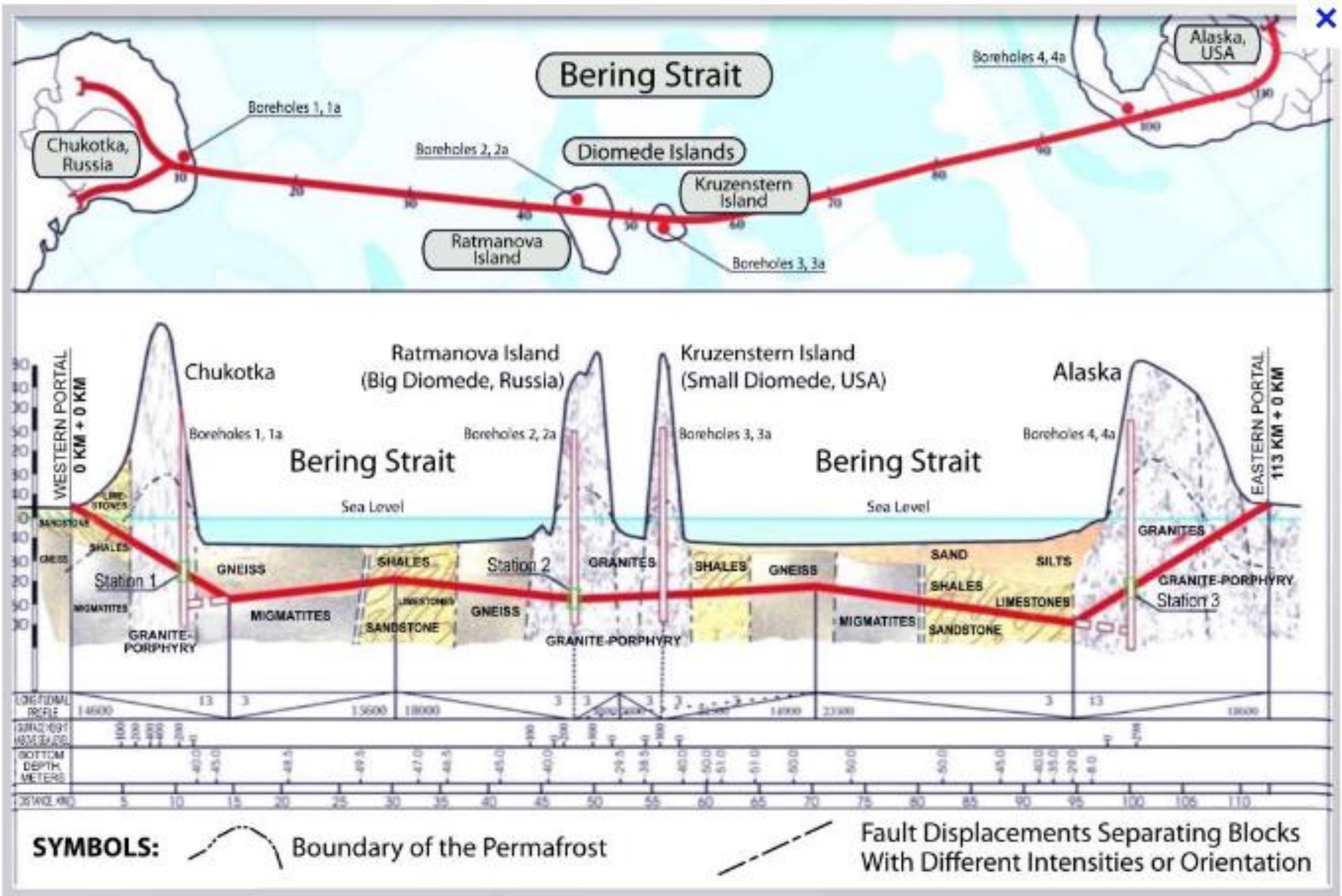
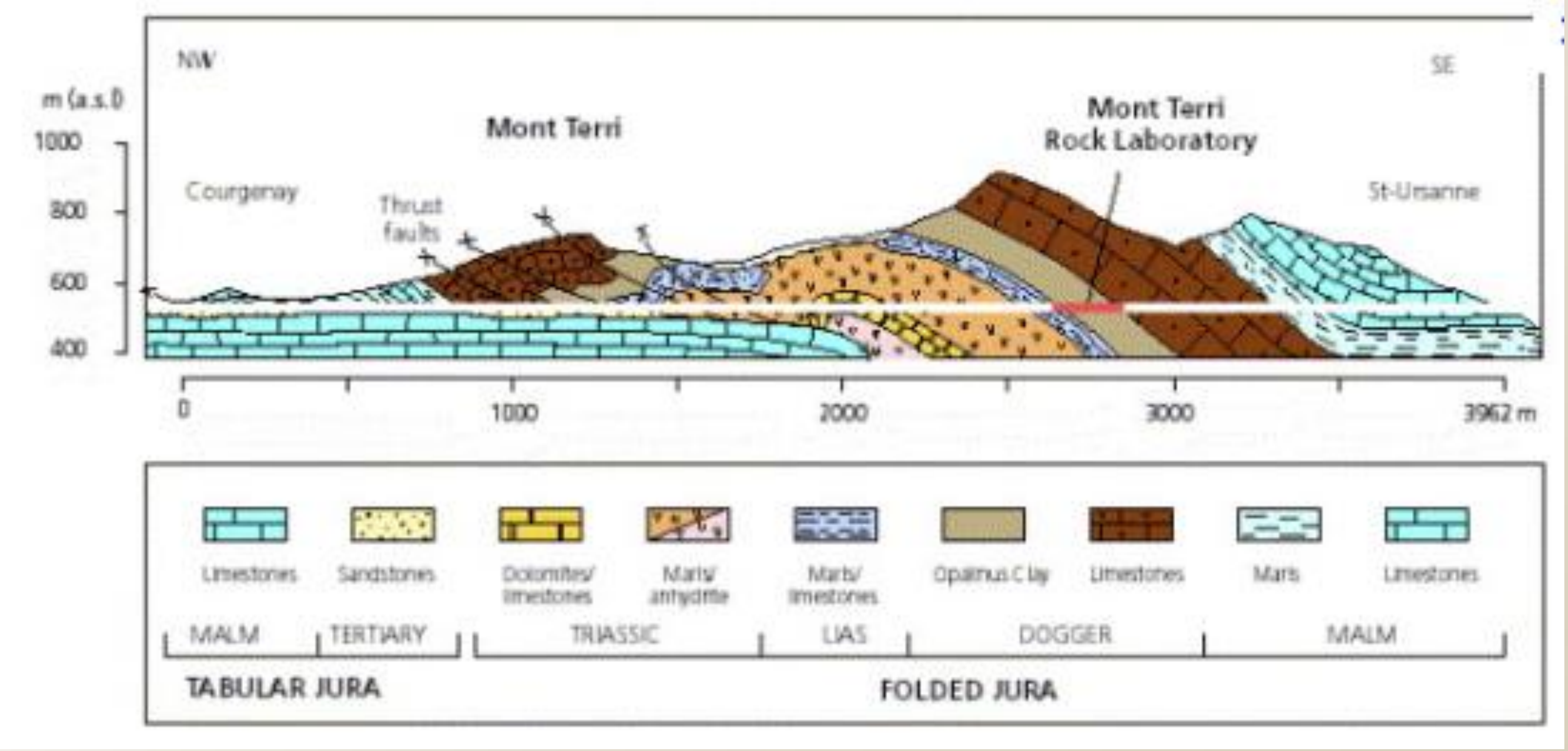
LESSON 1. INTRODUCTION



NORTHBOUND TUNNEL
Concept Design – Indicative Cross-section
LEGEND
Fill
Alluvium (silt, clay etc)
Alluvium (sand and gravel)
Aspley Formation
Rhyolitic Tuff
Claystone, Sandstone, Shale
Breccia
Neranleigh Fernvale Beds
Brisbane Tu
Horizontal scale 8:1
Ipswich Road
Pacific Motorway
WOOLLOONGABBA
South East Busway
Stanley Street
Vulture Street
Main Street
Shafston Avenue connection
Main Street
KANGAROO POINT
Captain Burke Park
BRISBANE RIVER
Ivory Street
Ann Street
Wickham Street
St Pauls Terrace
Water Street
FORTITUDE VALLEY
RNA Showgrounds
O'Connell Terrace
Campbell Street
BOWEN HILLS


Bering Strait
Chukotka, Russia
Boreholes 1, 1a
Boreholes 2, 2a
Diomede Islands
Kruzenstern Island
Ratmanova Island
Boreholes 3, 3a
Boreholes 4, 4a
Alaska, USA
Chukotka
Ratmanova Island (Big Diomede, Russia)
Kruzenstern Island (Small Diomede, USA)
Alaska
WESTERN PORTAL 0 KM + 0 KM
EASTERN PORTAL 113 KM + 0 KM
Bering Strait
Sea Level
Bering Strait
Sea Level
LIME-STONES
SANDSTONE
SHALES
GNEISS
Station 1
MIGMATITES
GRANITE-PORPHYRY
GNEISS
MIGMATITES
SHALES
LIMESTONES
SANDSTONE
GNEISS
Station 2
GRANITES
GRANITE-PORPHYRY
SHALES
GNEISS
MIGMATITES
SAND
SILTS
SHALES
LIMESTONES
SANDSTONE
GRANITES
GRANITE-PORPHYRY
Station 3
LONGITUDINAL PROFILE
BOTTOM DEPTH, METERS
DISTANCE, KM
SYMBOLS:
Boundary of the Permafrost
Fault Displacements Separating Blocks With Different Intensities or Orientation


NW
SE
m (a.s.l)
1000
800
600
400
Mont Terri
Mont Terri
Rock Laboratory
Courgenay
Thrust
faults
St-Ursanne
0
1000
2000
3000
3962 m
Limestones
Sandstones
Dolomites/
limestones
Marls/
anhydrite
Marls/
limestones
Opalinus Clay
Limestones
Marls
Limestones
MALM
TERTIARY
TRIASSIC
LIAS
DOGGER
MALM
TABULAR JURA
FOLDED JURA

Two subsea tunnels totaling 20 km in length have been given the green light by the Norwegian Government

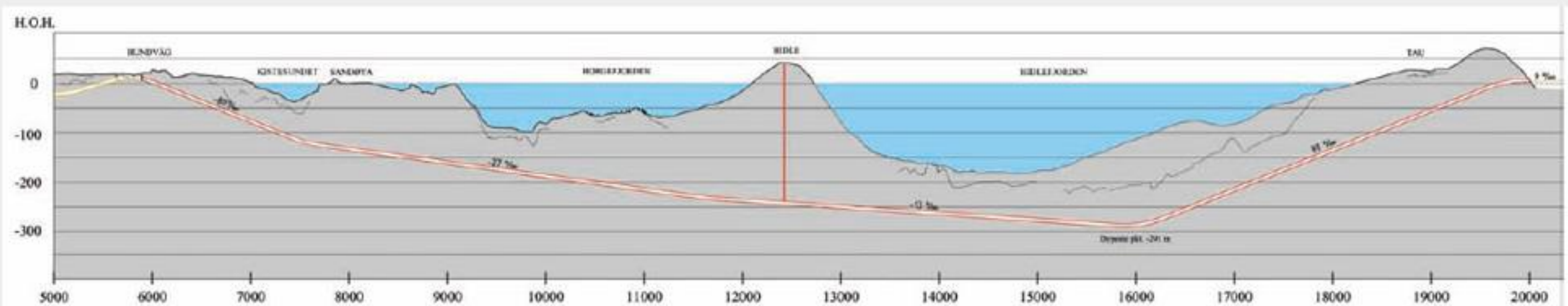
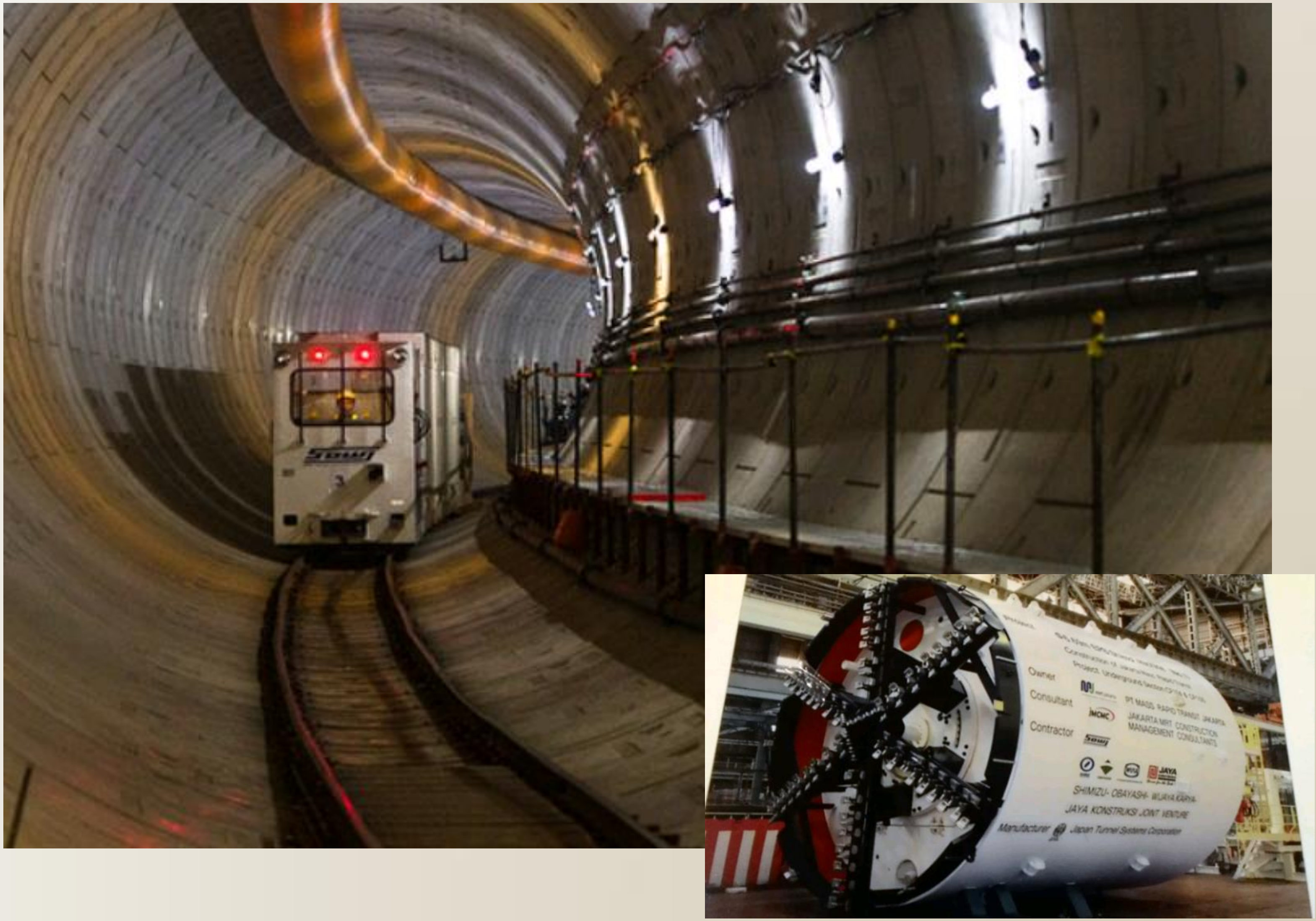
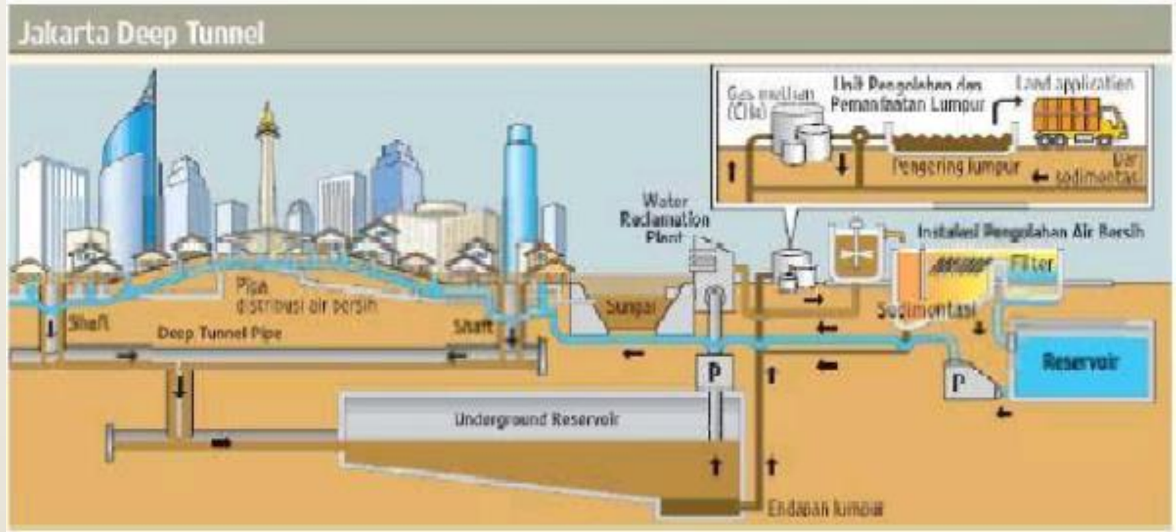


Fig 3. Hundvåg tunnel cross section

Mass Rapid Transit (MRT)



Jakarta Deep Tunnel



Benefits from tunneling

- The necessity for tunnels and the benefits they bring cannot be overestimated.
- Tunnels improve connections and shorten lifelines. Moving traffic underground, they improve the quality of life above ground and may have enormous economic impact.
- The utilization of underground space for storage, power and water treatment plants, civil defense and other activities is often a must in view of limited space, safe operation, environmental protection and energy saving. Of course, the construction of tunnels is risky and expensive and requires a high level of technical skill.

The oldest tunnels

- Europalinos tunnel (Samos, 500 BC) →1km
- Urner Loch (1707, first alpine tunnel in Switzerland) →64 m
- Mont-Cenis (France - Italy, 1857-1870, also called Fr´ejus tunnel) →12 km
- St. Gotthard rail tunnel (Switzerland, 1872-1878) →15 km

The longest tunnels

- Seikan (Japan, 1981-1984) → 54 km
- Euro-tunnel (France - England, 1986-1993) → 50 km
- Simplon I (Switzerland - Italy, 1898-1906) → 20 km
- Grand-Apennin (Italy, 1921-1930) →19 km
- New Gotthard (Switzerland, 1969-1980) →16 km

Large metro systems

- New York → 221 km
- London → 414 km
- Paris → 165 km
- Moscow → 254 km

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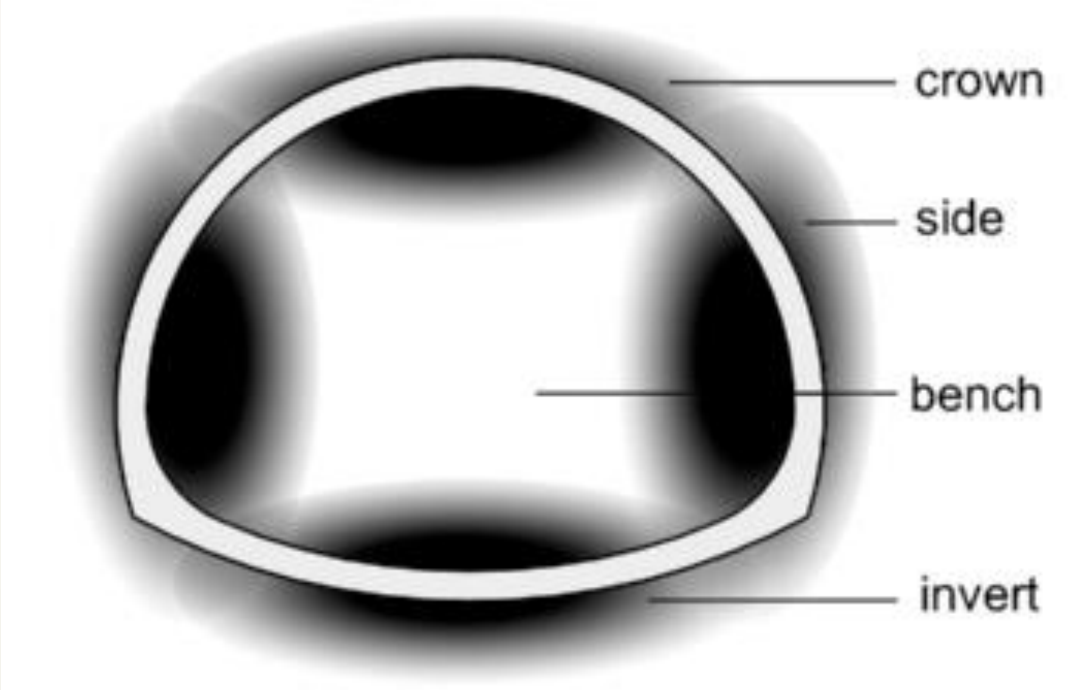
Notations in tunnelling



Parts of a tunnel cross section

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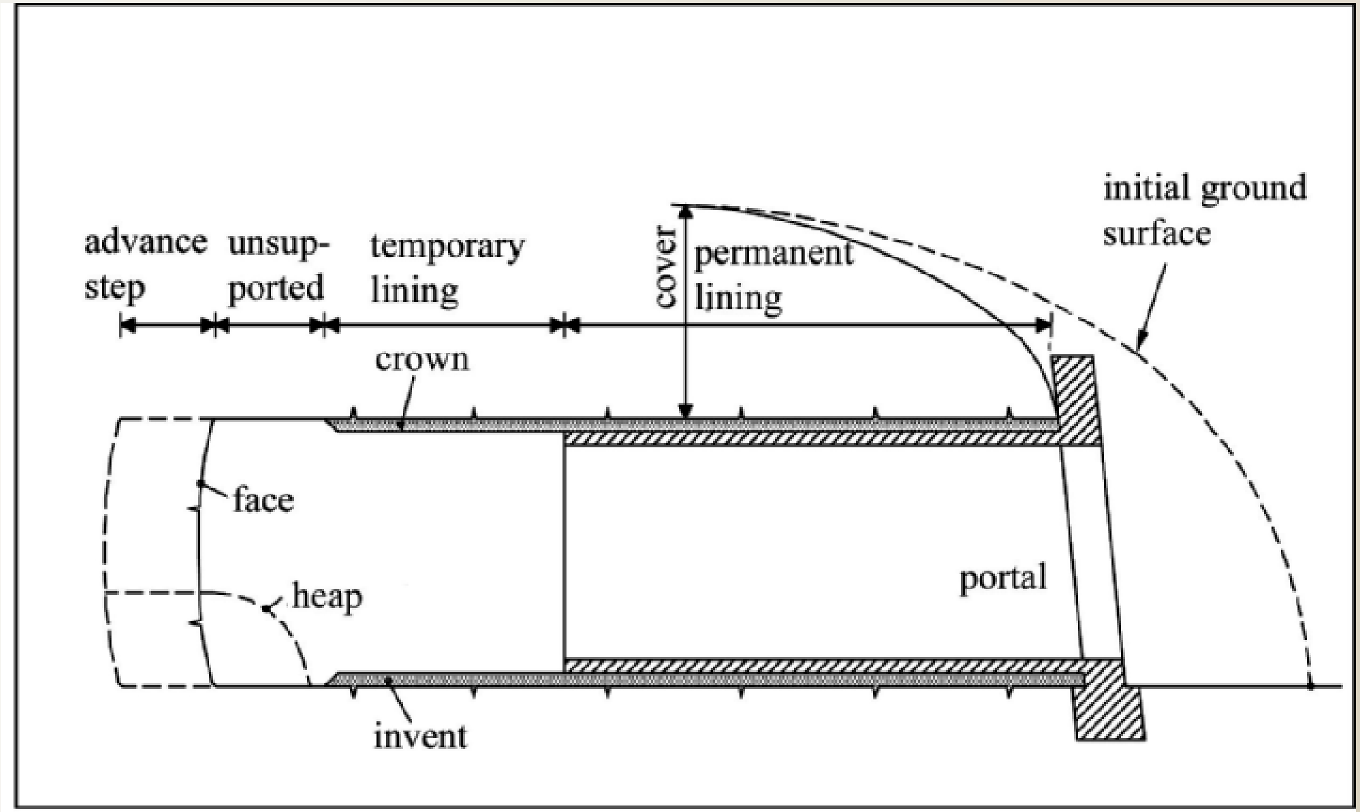


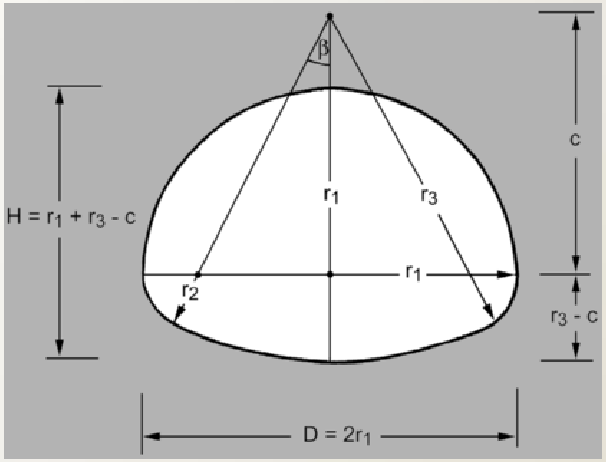
Longitudinal sections of heading

Cross sections

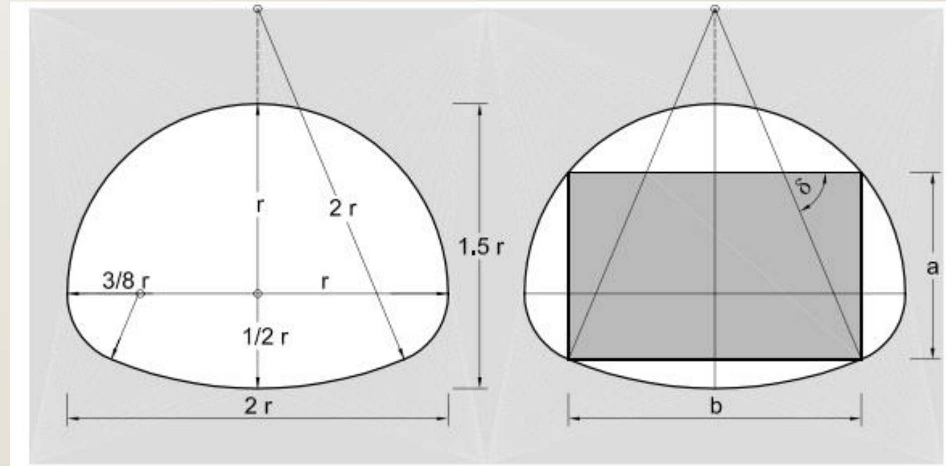
- The shape of a tunnel cross section is also called profile. Various profiles are conceivable, e.g. rectangular ones. The most widespread ones, however, are circular and (mostly oblate) mouth profiles).
- The choice of the profile aims at accommodating the performance requirements of the tunnel. Moreover it tries to minimise bending moments in the lining (which is often academic, since the loads cannot be exactly assessed) as well as costs for excavation and lining.
- Further aspects for the choice of the profile are: ventilation, maintenance, risk management and avoidance of claustrophobia of users.

Cross sections

- A mouth profile is composed of circular sections.
- The ratio of adjacent curvature radiuses should not exceed 5 ($r1/r2 < 5$)



Geometry of a mouth profile



Example of a mouth profile

The following relations refer to geometrical properties of mouth profiles. With the initial parameters r_1, r_2, r_3, it is obtained by formulas as follows:

$$\sin\beta = \frac{r_1 - r_2}{r_3 - r_2}$$

$$c = \sqrt{r_3^2 - 2r_2(r_3 - r_1) - r_1^2}$$

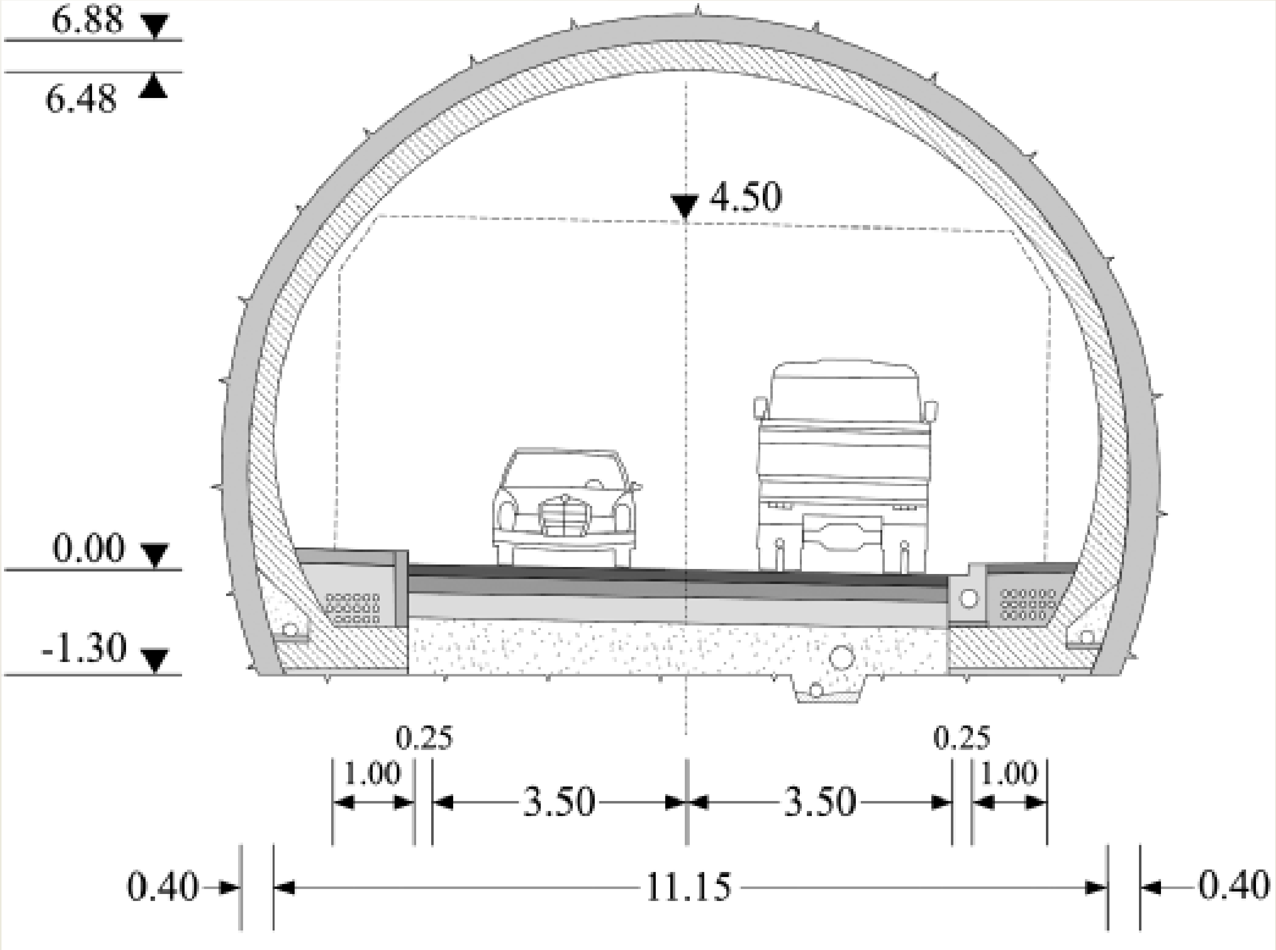
$$\text{cross section area } A = \frac{\pi}{2} r_1^2 + \left(\frac{\pi}{2} - \beta\right) r_2^2 + \beta r_3^2 - (r_1 - r_2)c \; ; \qquad (\beta \text{ in rad!})$$

$$\text{height } \; H = r_1 + r_3 - c$$
$$\text{span } \; D = 2r_1$$

A typical problem of tunnel design is to fit a rectangle into a mouth profile i.e. to choose r_1, r_2, r_3 in such a way that the mouth profile comprises the rectangle.

Typical tunnel cross sections Area	(m^2)
sewer	10
hydropower tunnels	10 - 30
motorway (one lane)	75
rail (one track)	50
metro (one track)	35
high speed rail (one track)	50
high speed rail (two tracks)	80 - 100

Hochrheinautobahn – B¨urgerwaldtunnel, A 98 Waldshut-Tiengen



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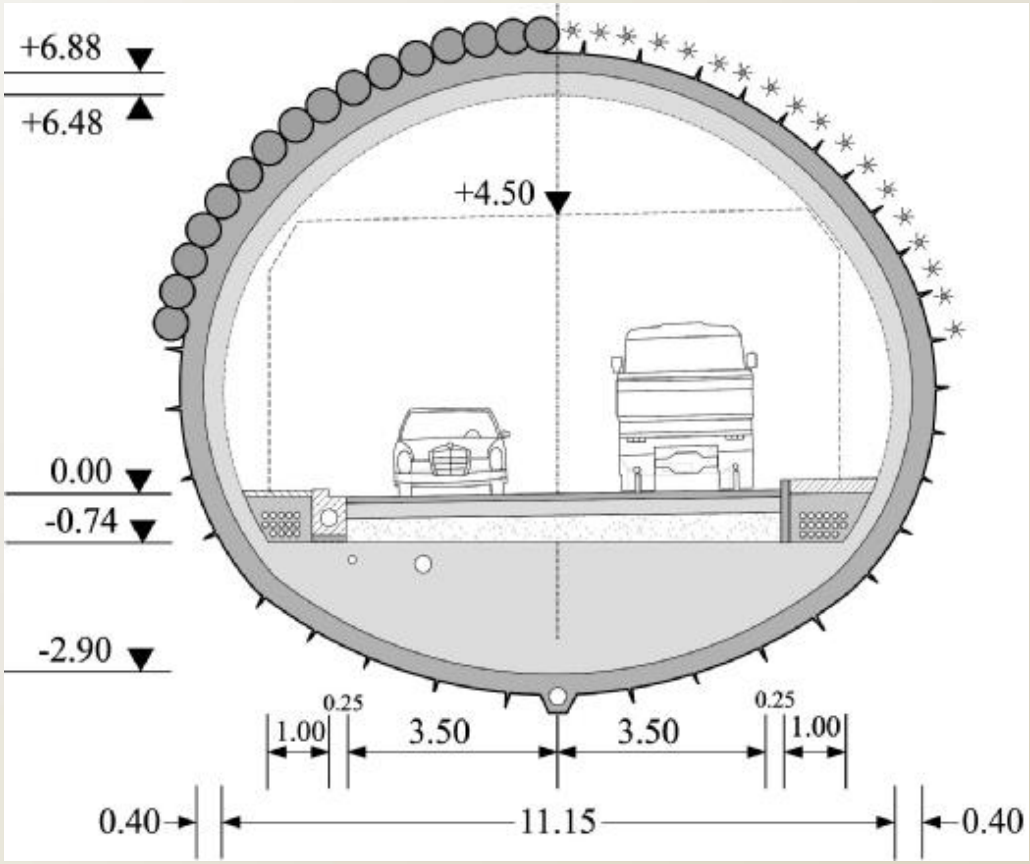
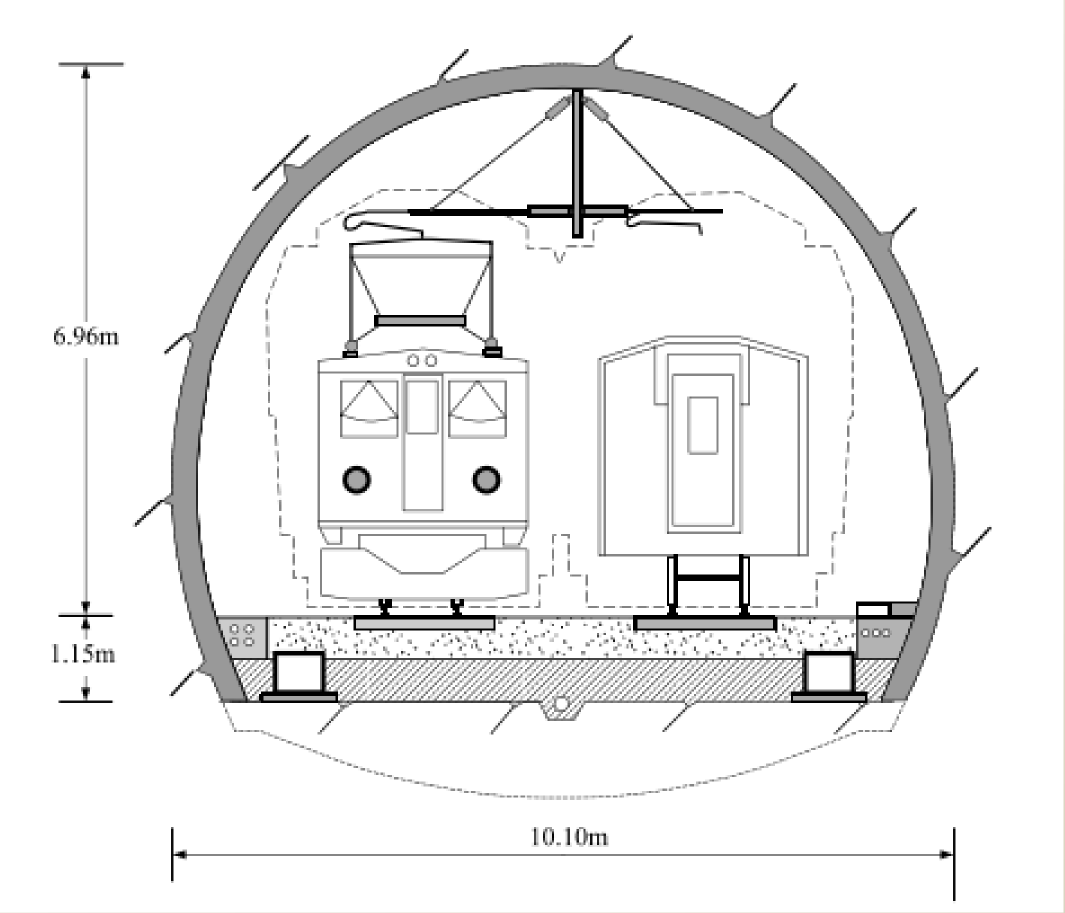
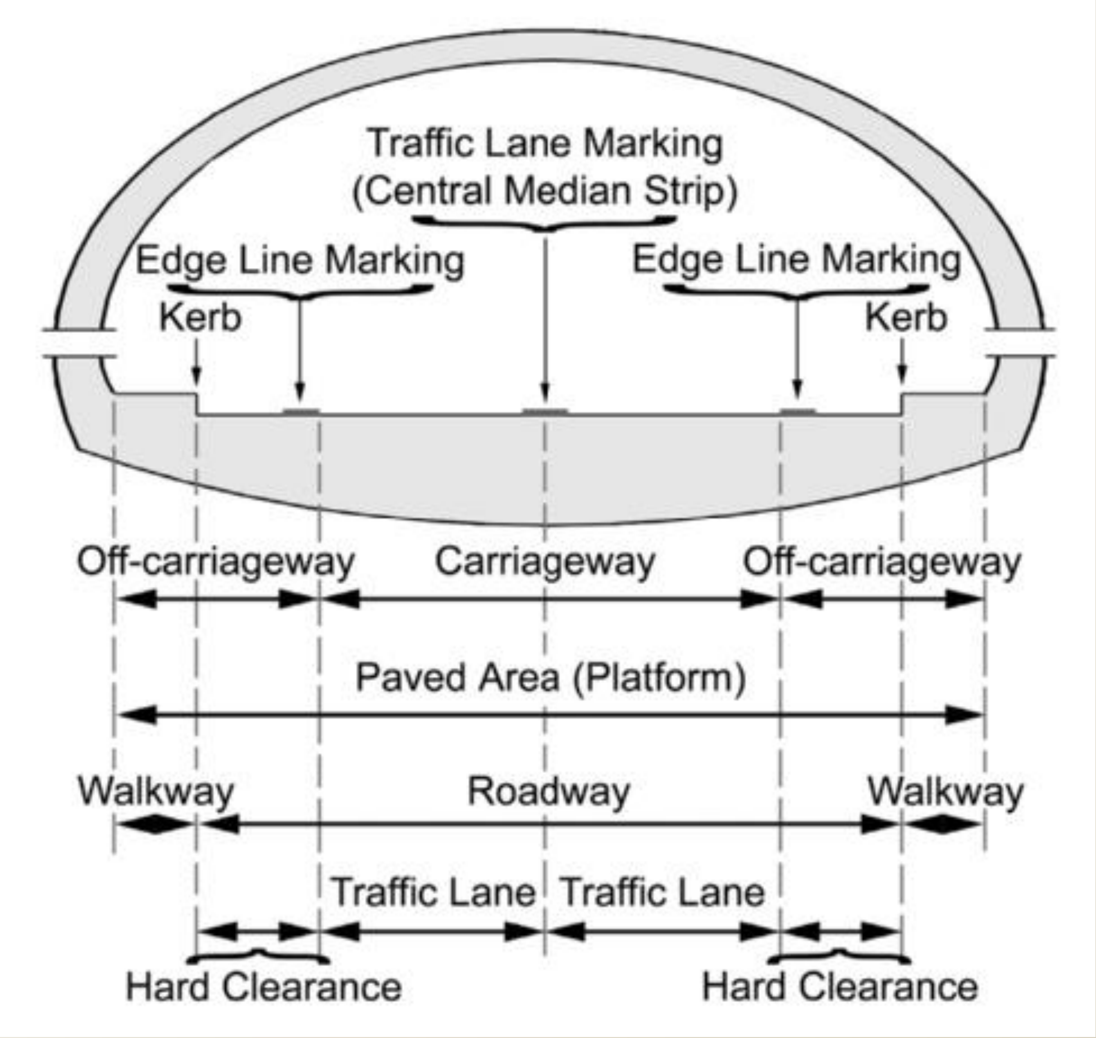


Hochrheinautobahn – B¨urgerwaldtunnel, A 98 Waldshut-Tiengen



Vereina Tunnel South, R¨atische Bahn; two tracks

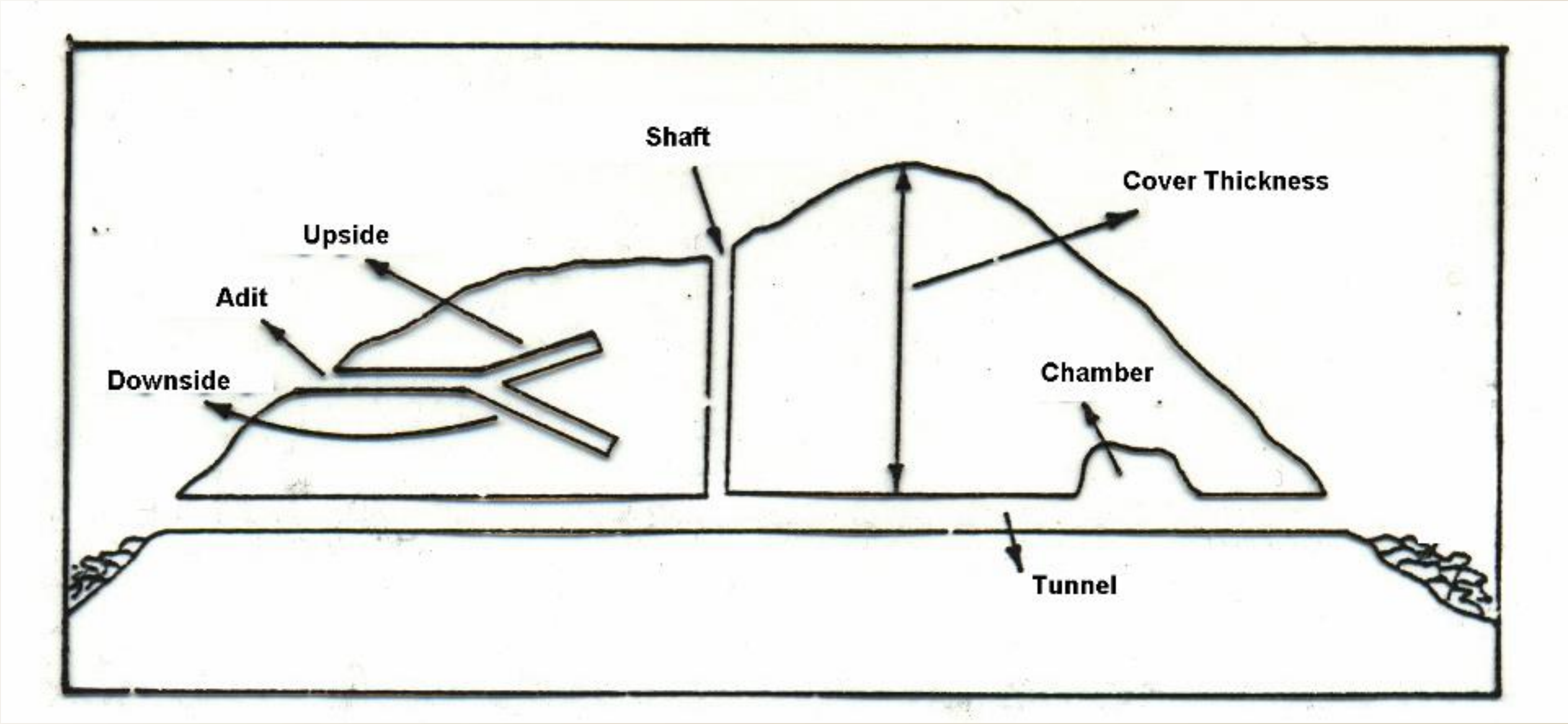
Road tunnels



TUNNEL DESCRIPTION

1. Made into natural material (rocks)
2. Empty inside
3. Carry the loads itself
4. Both ends are open to atmosphere
5. Generally horizontal
6. Thick walled structure looks like cylinder

Opening in mine



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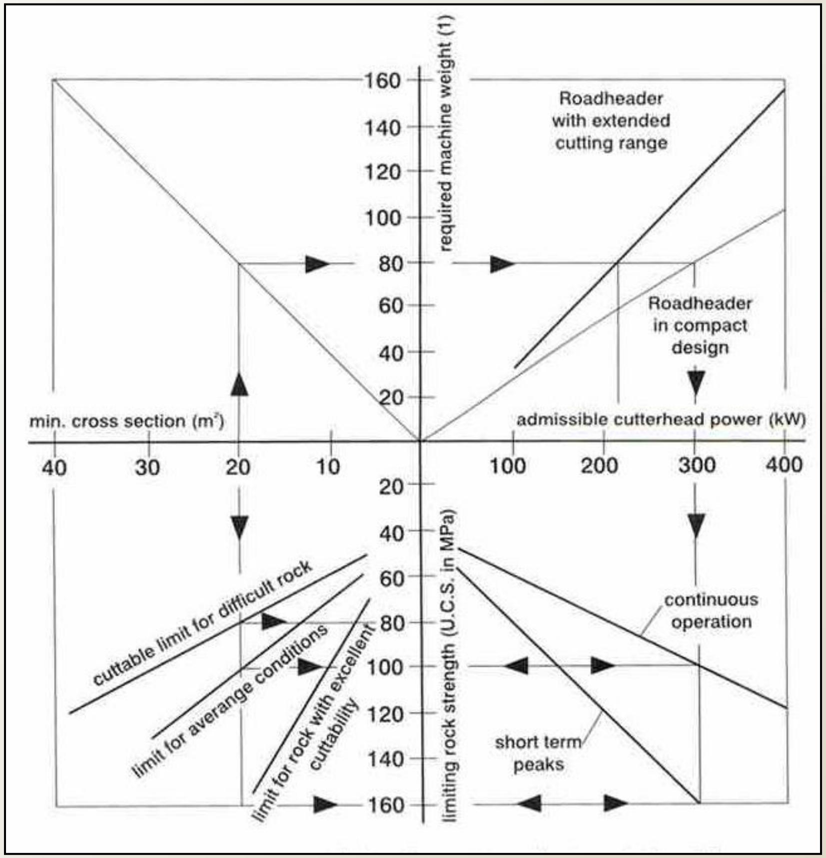
Equipment for Tunnelling

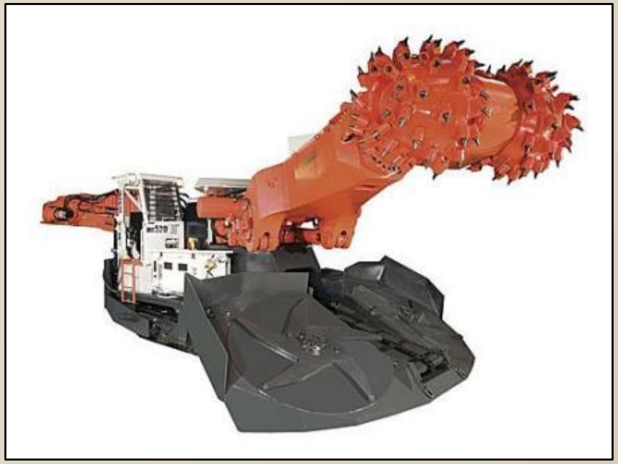
Continuous miner

Robbins Mobile Miner

Pengeboran dan Penggalian



Road header



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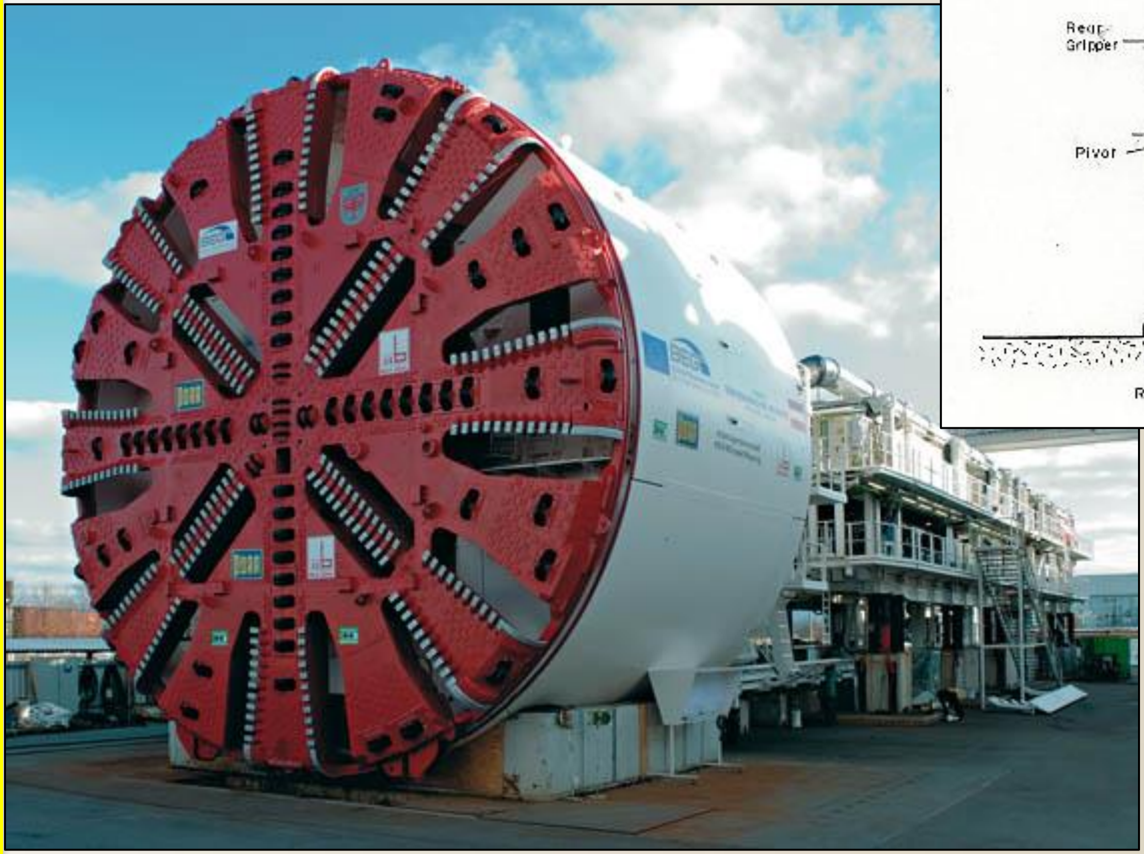
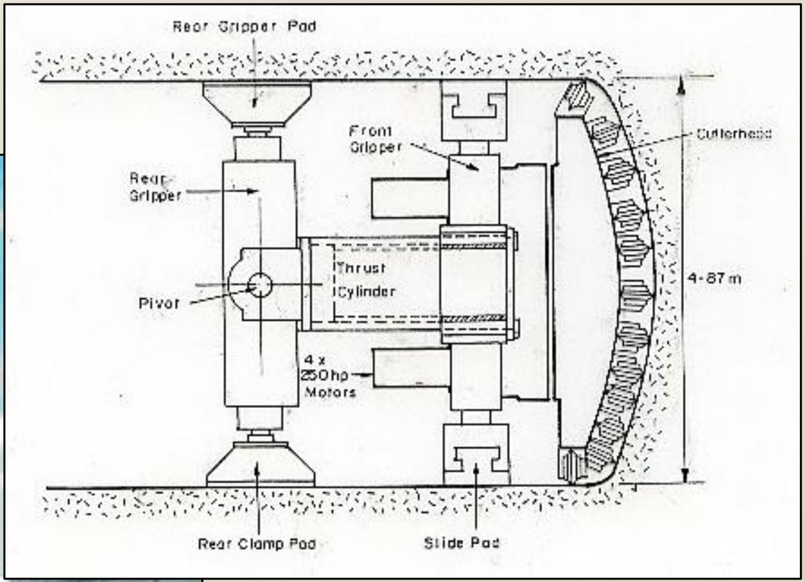
Tunnelling Boring Machine

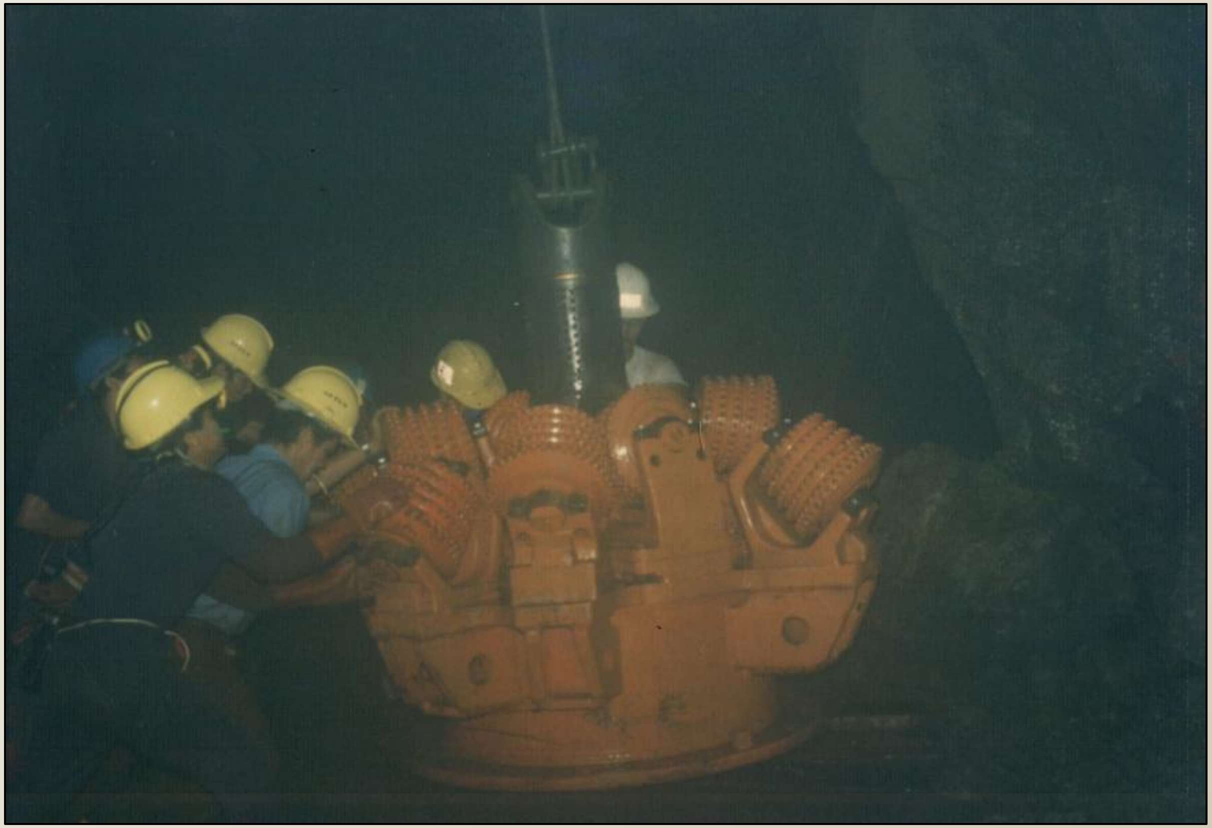




Tunnel boring machine

Equipment for Tunnelling

Raise boring machine



LESSON 2. GEOLOGICAL ASPECT OF TUNNELLING

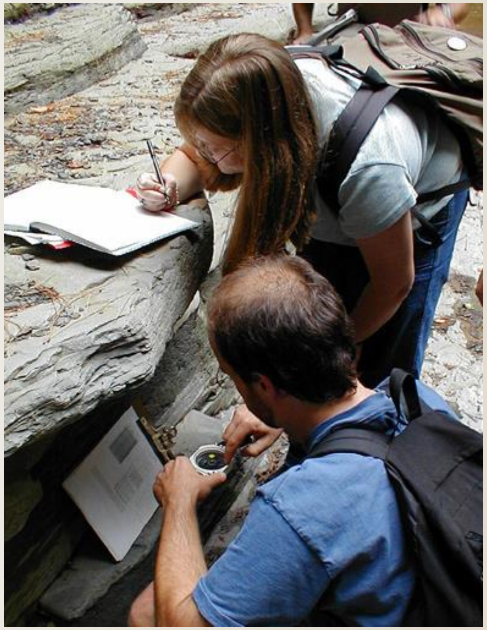
Introduction

- A rock mass is rarely continuous, homogeneous or isotropic. It is usually intersected by a variety of discontinuities such as faults, joints, bedding planes, and foliation.
- In addition, there can be a number of different rock types which may have been subjected to varying degrees of alteration or weathering.
- It is clear that the behaviour of the rock mass, when subjected to the influence of mining excavations, depends on the characteristics of both the rock material and the discontinuities.

The characterization of rock mass

- Descriptive indices required to fully characterise the rock mass comprise weathering/alteration, structure, colour, grain size, intact rock material compressive strength and rock type.
- The details of the discontinuities such as orientation, persistence, spacing, aperture/thickness, infilling, waviness and unevenness for each set.
- The resulting rock mass can be described by block shape, block size and discontinuity condition.
- An evaluation of the potential influence of groundwater and the number of joint sets, which will affect the stability of the excavation, completes the description.

Mapping of geological structure



Mapping of geological structure is an essential component of the design of underground excavations.

Structural planes run through the rock mass and may divide it into discrete blocks of rock, which can fall or slide from the excavation boundary, when they are not adequately supported and when the stress conditions are favourable for structural failure.

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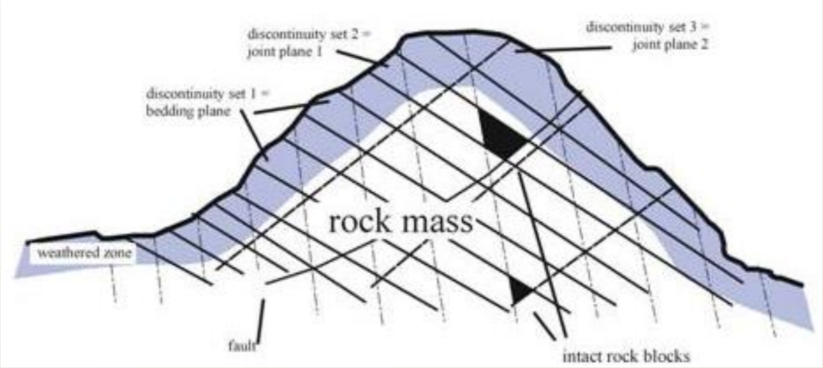
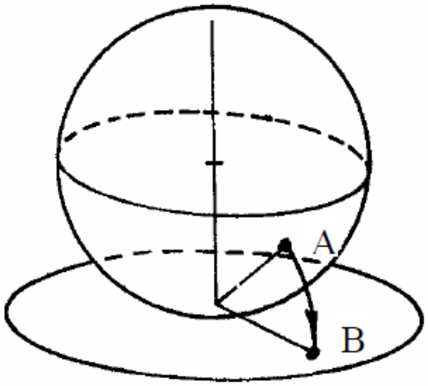


Three examples of typical engineering geological descriptions are:

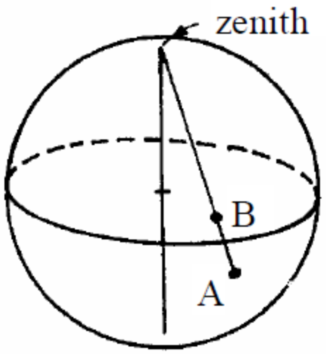
- Slightly weathered, slightly folded, blocky and schistose, reddish grey, medium grained, strong, Garnet-mica schist with well developed schistosity dipping 45/105. Schistosity is highly persistent, widely spaced, extremely narrow aperture, iron stained, planar, and smooth. Moderate water inflow is expected.
- Slightly weathered, blocky, pale grey, coarse grained, very strong, Granite with three sets of persistent, widely spaced, extremely narrow, iron stained, planar, rough, wet joints.
- Fresh, blocky (medium to large blocks), dark greenish grey, coarse grained, very strong Norite with two sets of persistent, widely to very widely spaced (600 mm), extremely narrow, undulating, rough, dry joints.

Structural geological data presentation

- A single plane oriented in three dimensional space is shown in Figure. The intersection of the plane with the reference sphere, shown in this figure as a shaded part ellipse, defines a great circle when projected in two-dimensional space.
- A pole is defined at the point where a line, drawn through the centre of the sphere perpendicular to the plane, intersects the sphere.

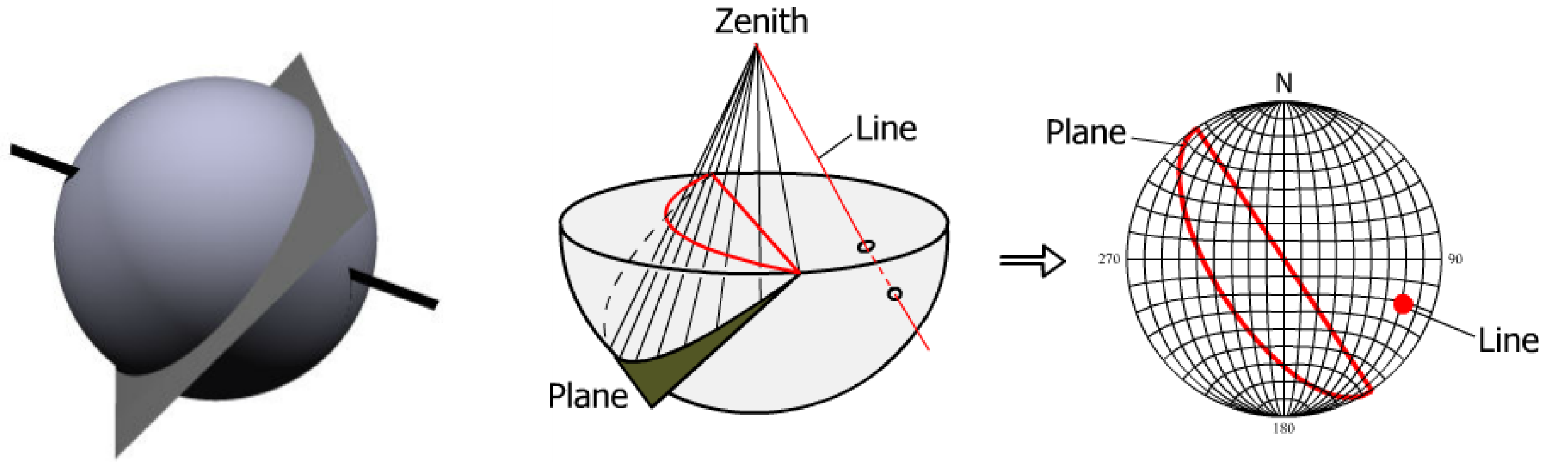


Equal area projection



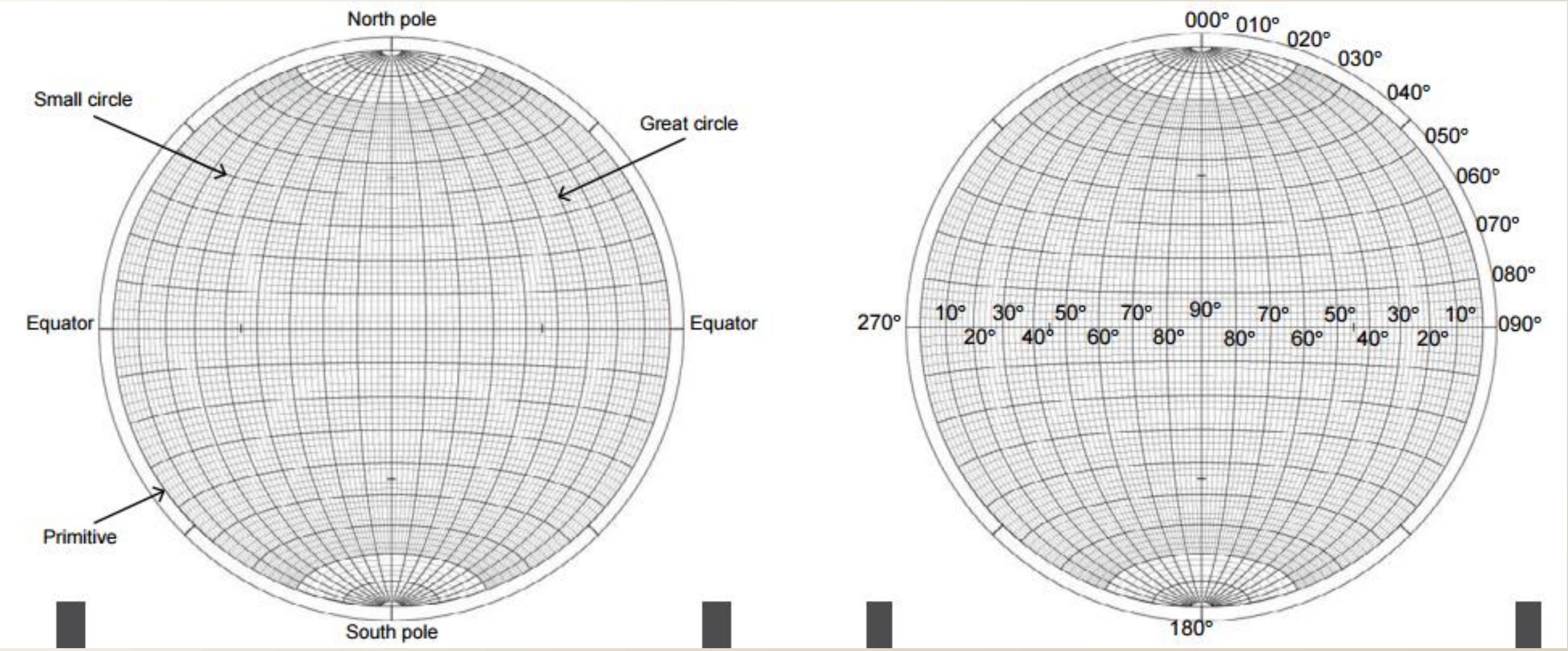
Equal angle projection

The Projection



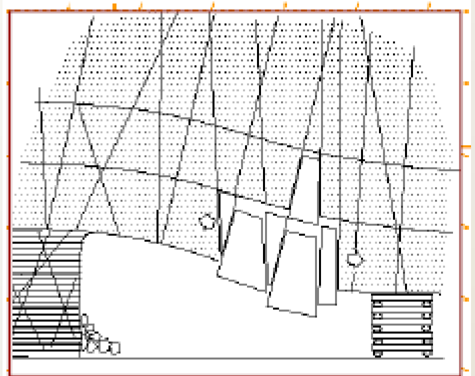
3-D illustration   sphere projection   2-D equal area projection

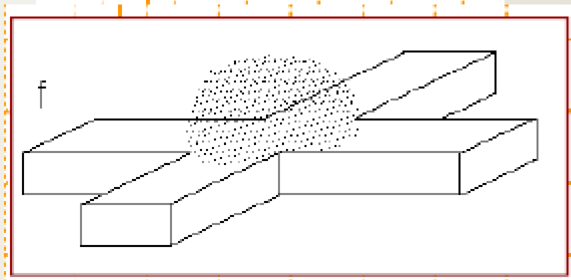
Stereonet terminology

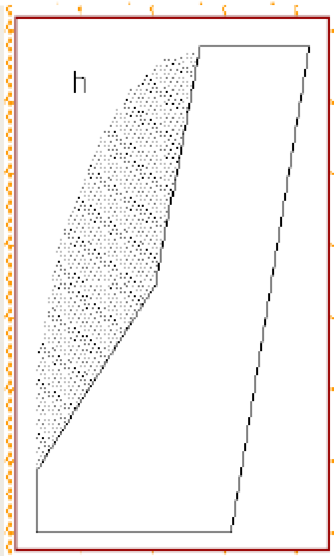


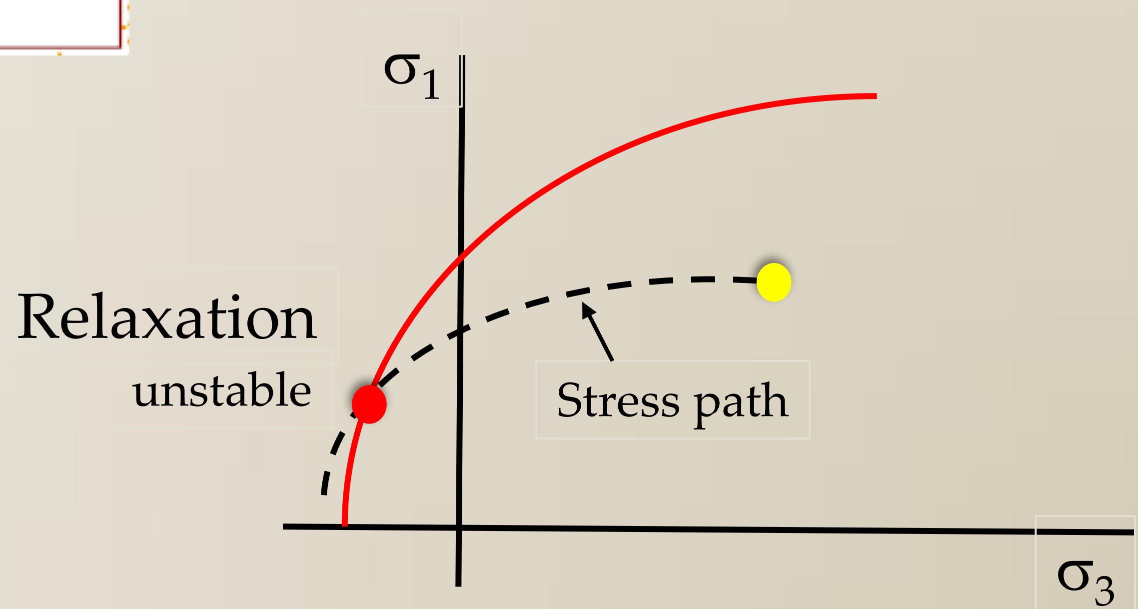
Structurally-Controlled Instability Mechanisms

Underground Instability Mechanisms



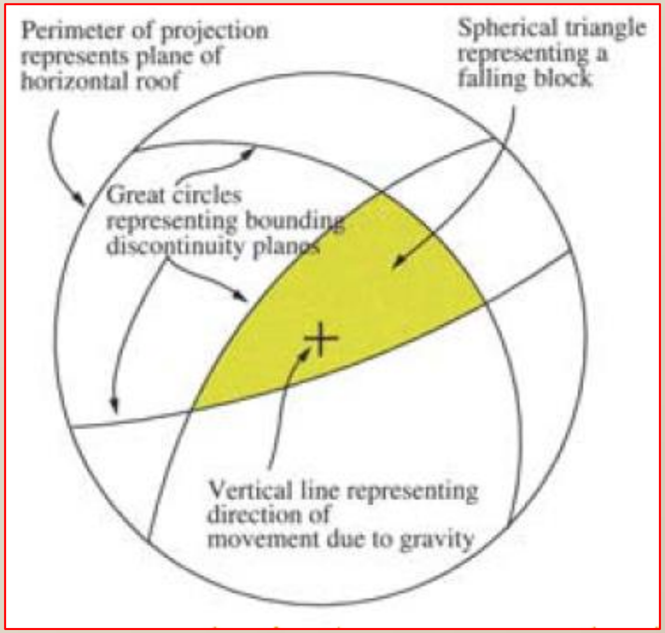






Kinematic Analysis – Underground Wedges

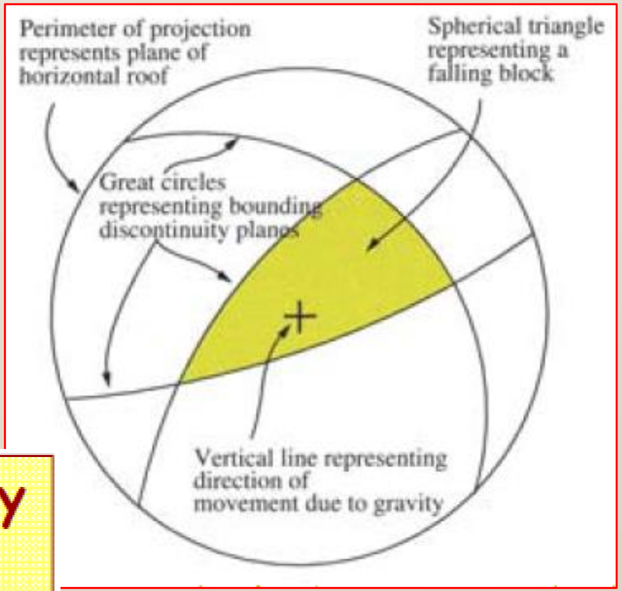
The minimum requirement to define a potential wedge is four non-parallel planes; the excavation periphery forms one of these planes. On a hemispherical projection, these blocks may be identified as spherical triangles where the plane of projection represents the excavation surface.



If a tetrahedral block/wedge exists, there are three kinematic possibilities to be examined: the block falls from the roof; the block slides (either along the line of maximum dip of a discontinuity, or along the line of intersection of two discontinuities); or the block is stable.

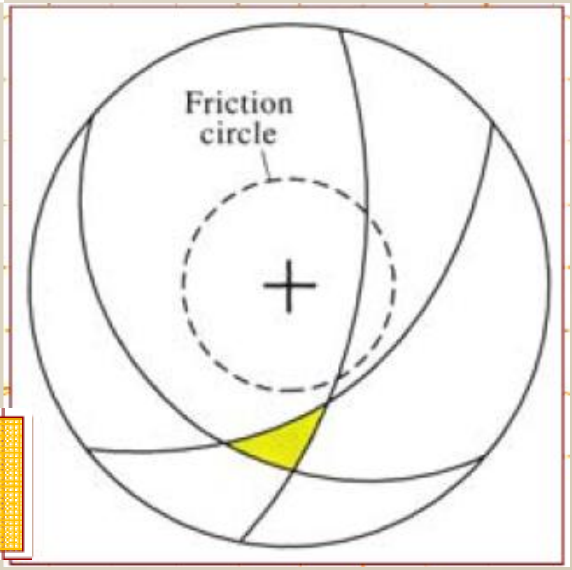
Analysis of Kinematic Admissibility - Falling

Falling occurs when a block detaches from the roof of an excavation without sliding on any of the bounding discontinuity planes. In the case of gravitational loading, the direction of movement is vertically downwards.



kinematically admissible

This is represented on the projection as a line with a dip of 90º, i.e. the centre of the projection. Thus, if this point falls within the spherical triangle formed by the bounding discontinuities, falling is kinematically admissible.



kinematically inadmissible = stable

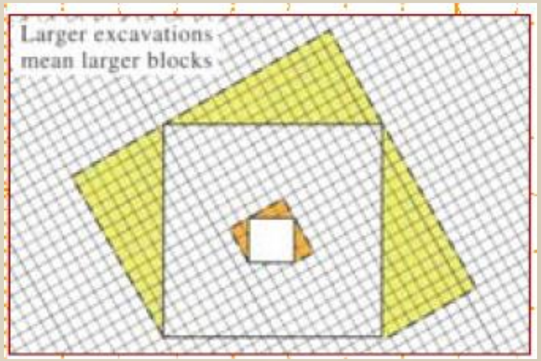
Geometrical Analysis of Maximum Wedge Volume

Once a series of joint sets have been identified as having wedge forming potential, several questions arise :

- **in the case of a falling wedge, how much support will be required to hold it in place (what kind of loads on the added support can be expected, how dense will the bolting pattern have to be, etc.);**

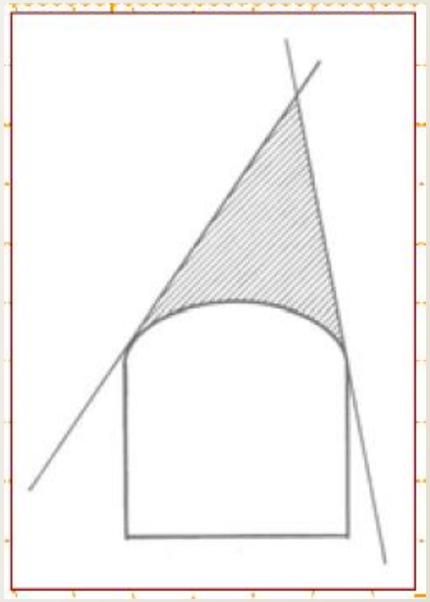
- **in the case of a sliding wedge, do the shear stresses exceed the shear strength along the sliding surface, i.e. that provided by friction and sometimes cohesion (in the form of intact rock bridges or mineralized infilling), and if so, how much support will be required to stabilize the block, how dense will the bolting pattern have to be, etc..**

In both cases, the volume/weight of the maximum wedge that may form is required. This can be determined through further geometrical constructions.



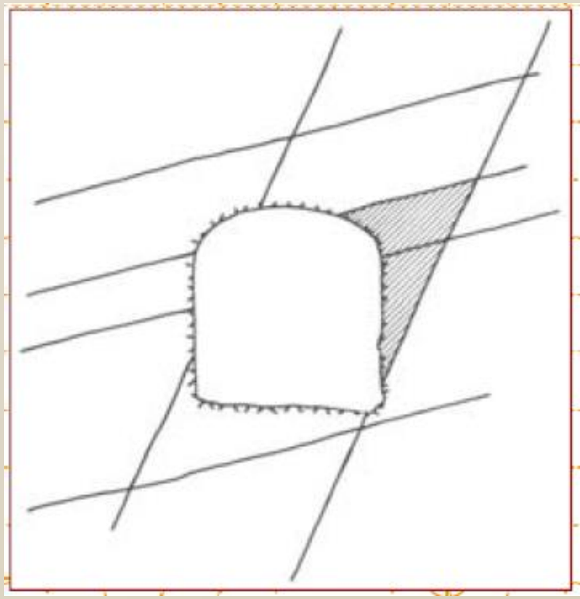
Maximum Wedge and Key Block Theory

Key block theory tries to build on wedge analysis by establishing a complete list of multiple blocks that may fail and a relative block failure likelihood distribution whose modes define the critical blocks.



… area of the maximum key-block for an underground opening.

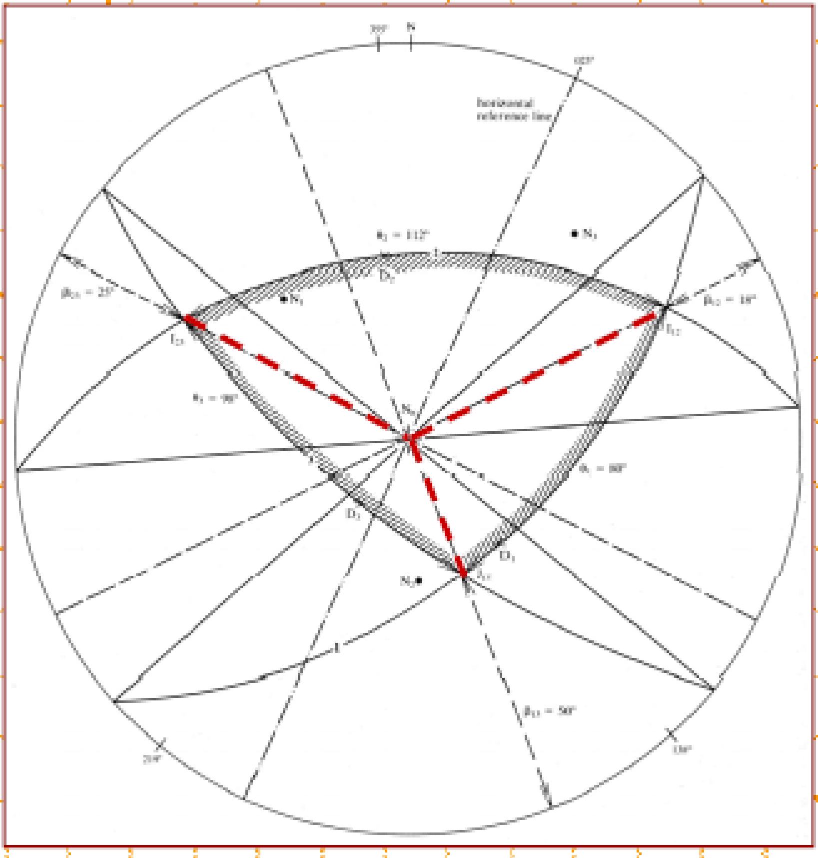
… "maximum wedge" formed by multiple key-blocks intersecting an excavation.



Goodman & Shi (1995)

Geometrical Analysis of Maximum Wedge Volume

To calculate the maximum wedge volume:



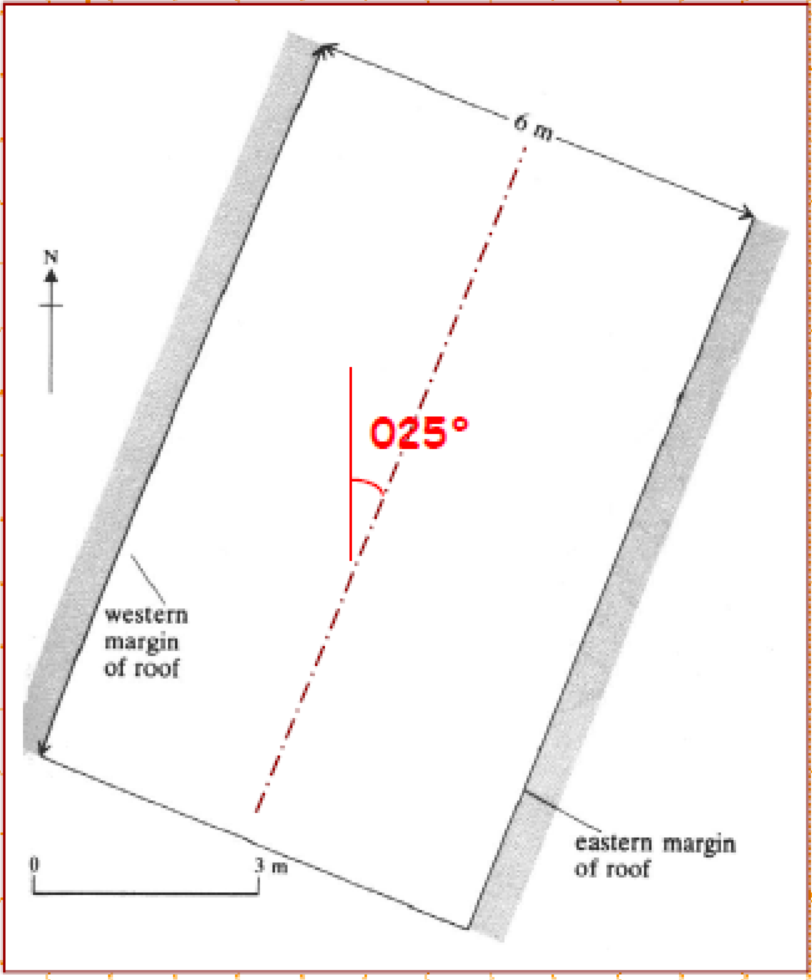
Priest (1985)

1) Identify the joint planes/great circles on the stereonet plot that form the wedge. In this example, the three persistent, planar discontinuity sets have dip directions/dips of: (1) 138/51, (2) 355/40, (3) 219/67.

   Together, these joints are known to form wedges within the horizontal, planar roof of an excavation in sedimentary rock.

   The stereonet construction is finished by drawing lines passing through the corners of the spherical triangle and centre of the stereonet.

Maximum Wedge Volume



2) On a separate sheet of paper, construct a scaled plan view, where the width of the window represents the width of the excavation. As such, the analysis will consider the largest block that could be released from the excavation roof.

In this particular example, the roof is rectangular in shape, is 6 m wide, and has it's long axis orientated at an azimuth of 025°.

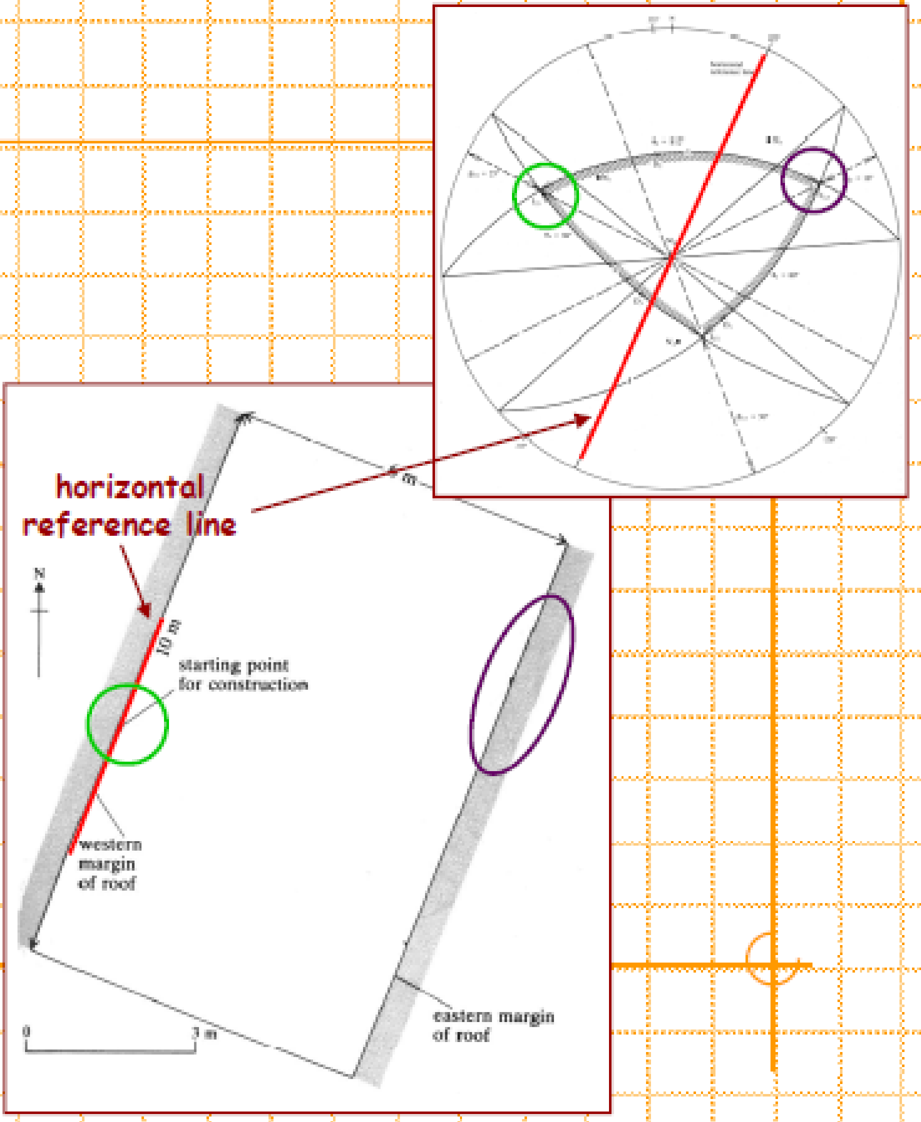
Given that the great circle representing the horizontal plane through the tunnel coincides with that of the stereonet projection, it is convenient to construct the window aligned parallel to the tunnel axis.

Maximum Wedge Volume

3) On the scaled window, mark an arbitrary horizontal reference line and starting point. For example, about halfway along the western margin of the roof.

Inspection of the spherical triangle in the stereonet plot suggests that the corner of the face triangle formed by planes 2 and 3 will touch the western margin of the roof, and the corner formed by planes 1 and 2 will touch the eastern margin when the largest possible tetrahedral block is considered.

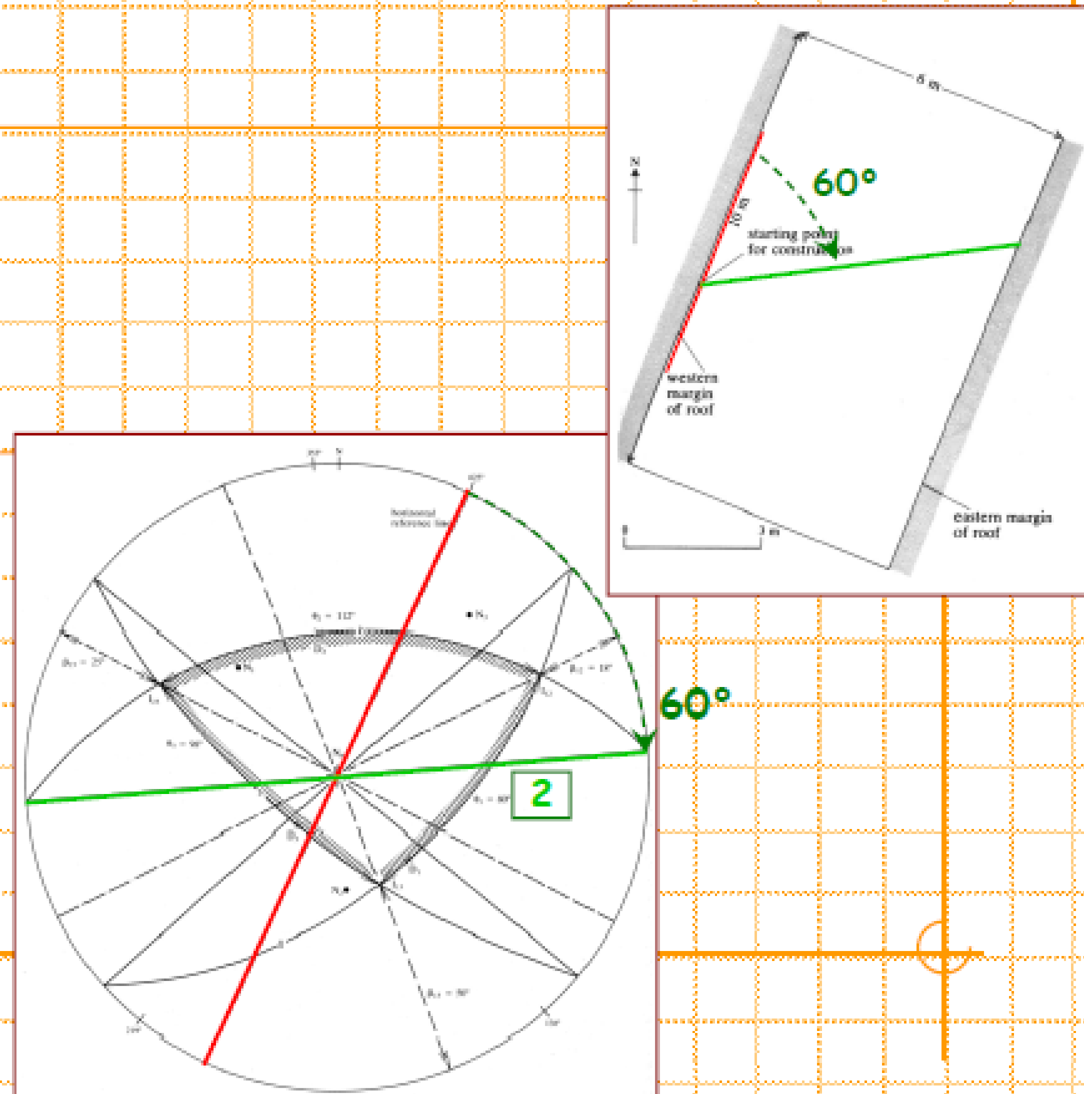
As such, the arbitrary reference point can represent the corner of the face triangle formed by planes 2 and 3.



Maximum Wedge Volume

4) The lines associated with planes 2 and 3 can now be added to the window construction by counting off the angles between the horizontal reference line on the stereonet plot (at 025°) and the diametral lines for planes 2 and 3 (striking at 085° and 129°, respectively).

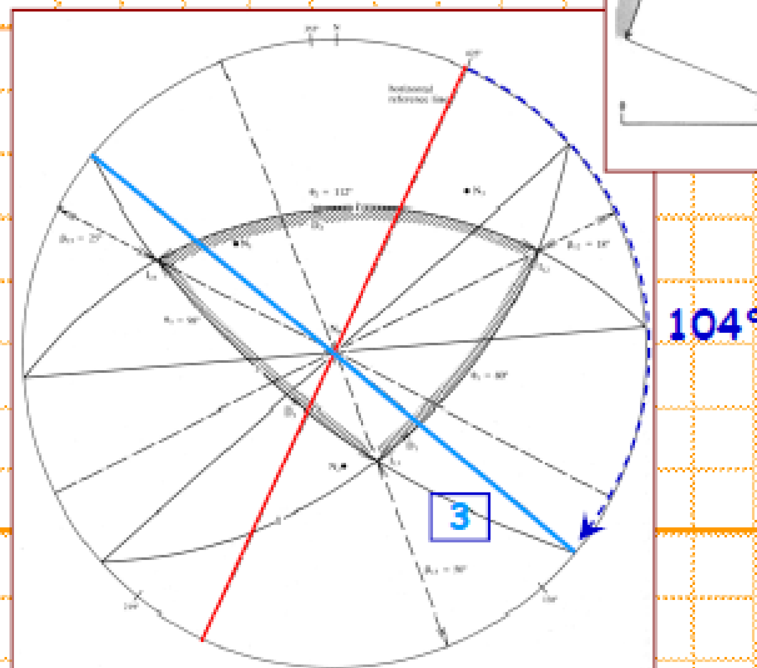
These angles can then be transferred to the window construction and measured off relative to the starting point and reference line along the western margin of the roof.

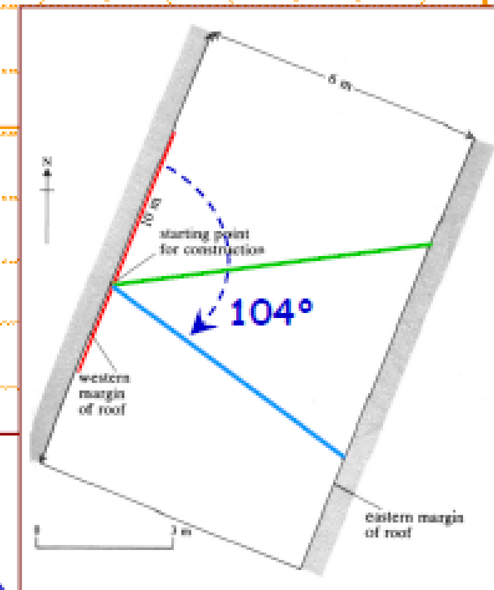


Maximum Wedge Volume

4) The lines associated with planes 2 and 3 can now be added to the window construction by counting off the angles between the horizontal reference line on the stereonet plot (at 025°) and the diametral lines for planes 2 and 3 (striking at 085° and 129°, respectively).

These angles can then be transferred to the window construction and measured off relative to the starting point and reference line along the western margin of the roof.

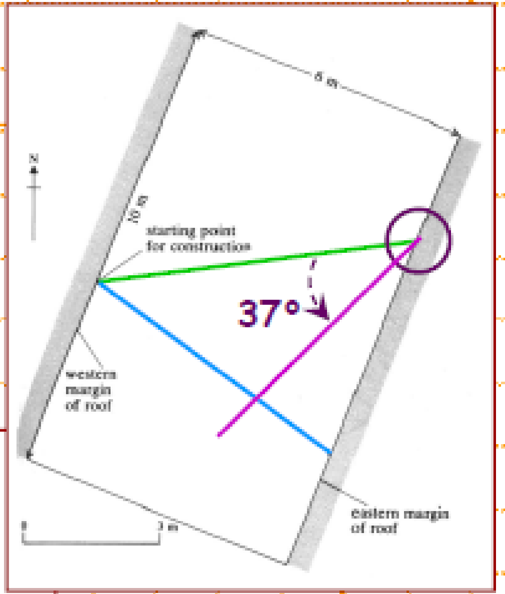


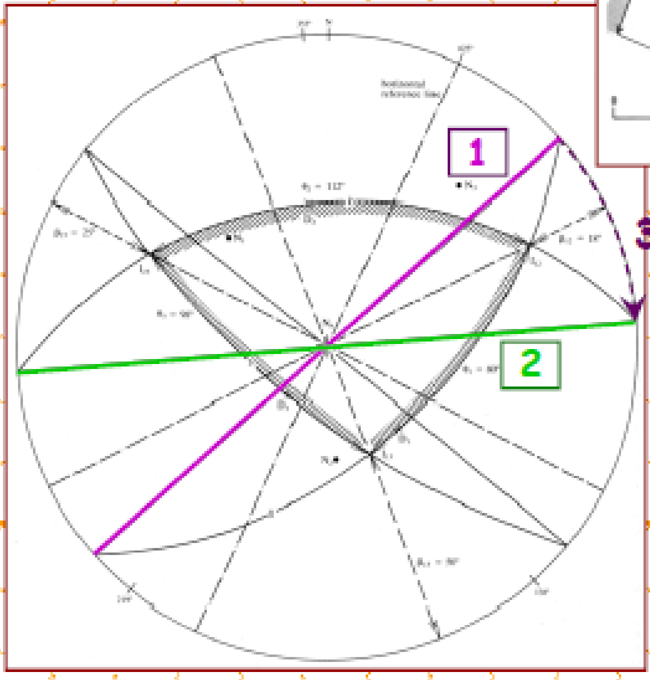


Maximum Wedge Volume

5) The point where the line for plane 2 intersects the eastern margin of the roof in the window construction represents the corner of the face triangle formed by planes 1 and 2. Thus, the line for plane 1 can be added by measuring the angle between the two planes on the stereonet and transferring it to the window construction.

The outline/trace of the wedge on the tunnel roof is now complete.

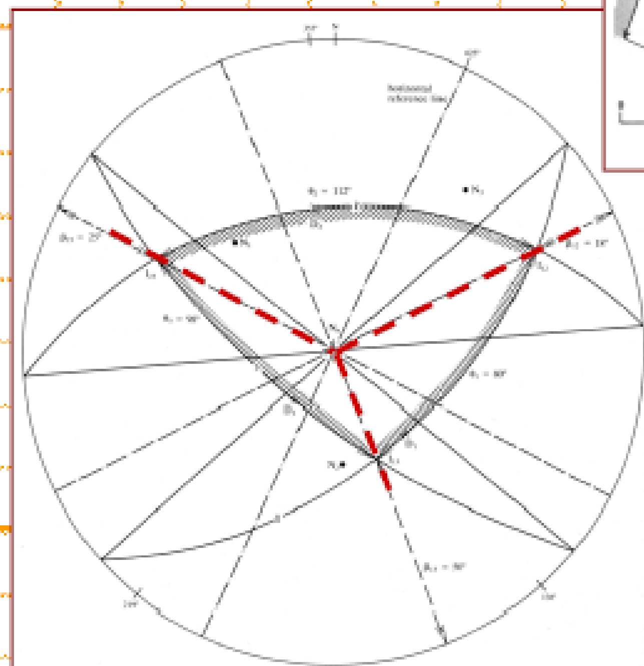


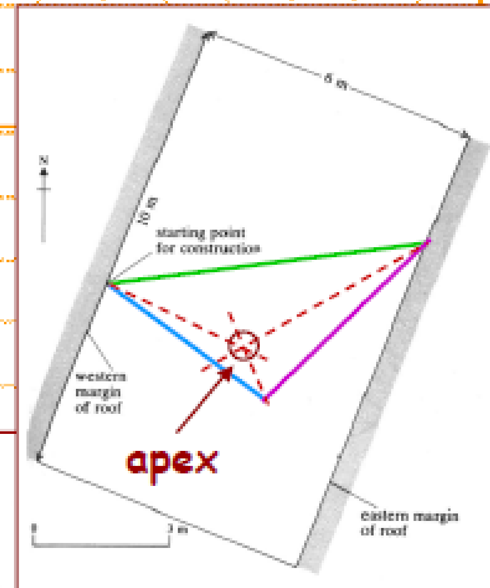


Maximum Wedge Volume

6) The next step is to add the corner edges of the wedge to complete the 3-D trace of the tetrahedron in the window construction box.

This can be done following a similar procedure by transferring the lines of intersection between the planes (i.e. I_{12}, I_{23}, I_{13}) and their measured angles from the stereonet to the window construction.

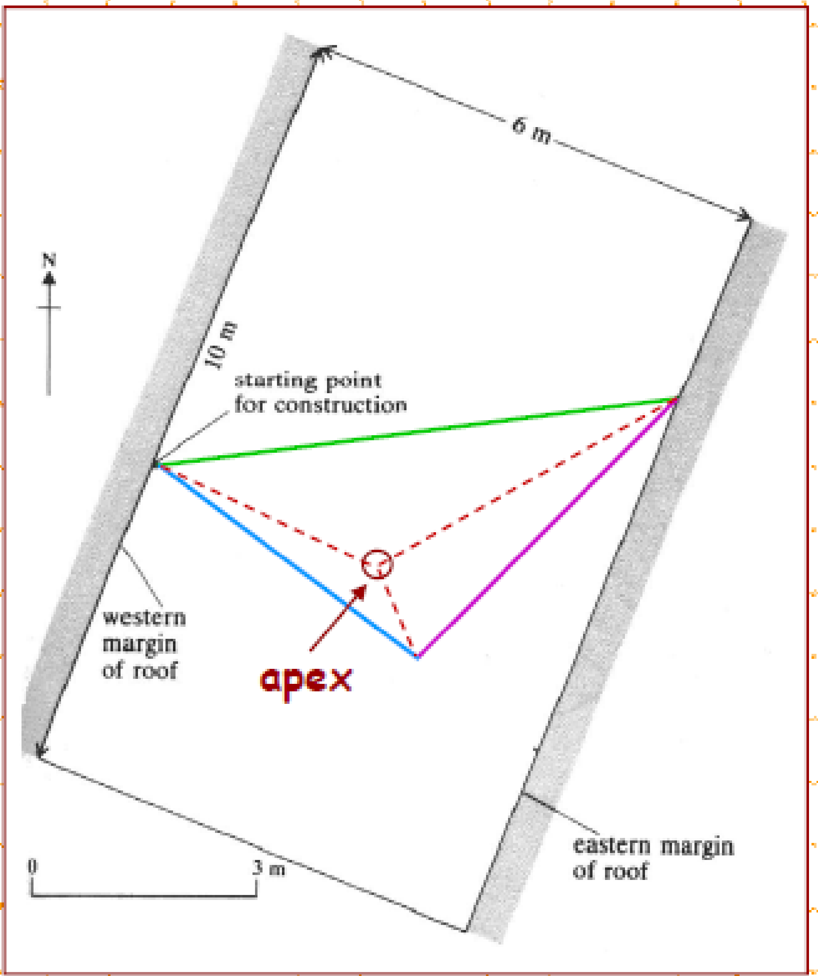




Maximum Wedge Volume

7) Since this construction can be completed graphically by overlaying the stereonet with the window construction, or geometrically by measuring the angles off the stereonet and transferring them onto the window construction, several checks can be made to find any errors that may have arisen.

The final step involving the finding of the location of the wedge's apex also gives a valuable check since the area of the triangle of error formed by these converging lines is a measure of any imprecision in the construction.



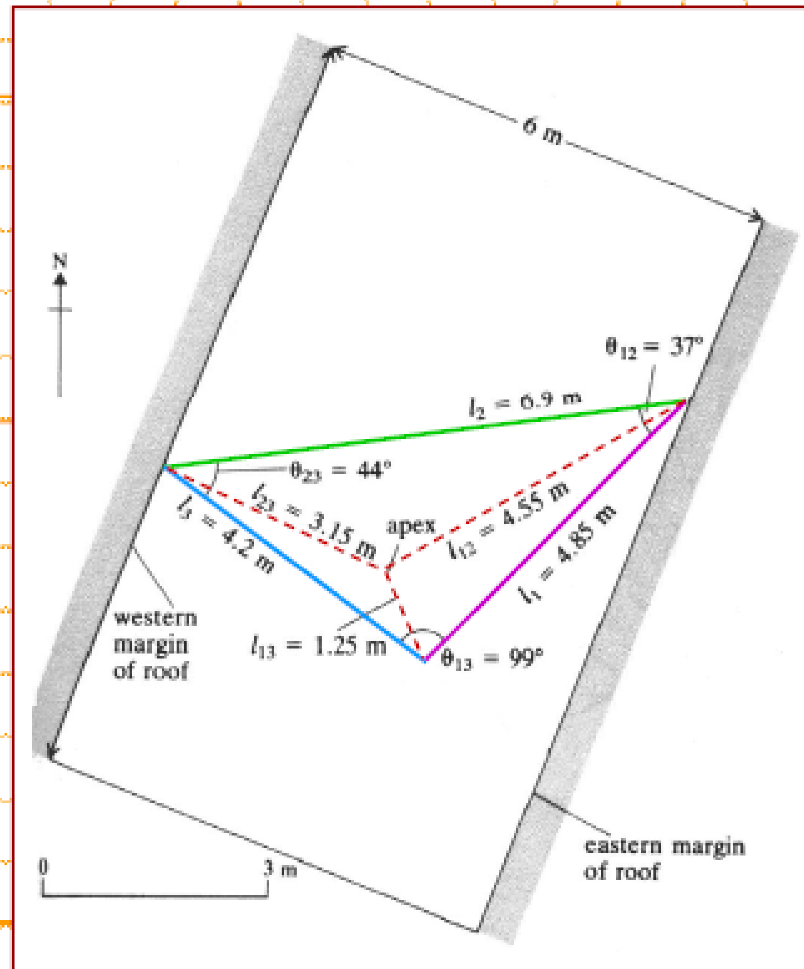
Priest (1985)

Maximum Wedge Volume

8) The dimensions of the face triangle appearing on the excavation surface can now be scaled off directly from the construction. It's area, A_f, can be found by taking any pair of adjacent sides and their included angles:

$$A_f = \tfrac{1}{2} l_1 l_2 \sin\theta_{12} = \tfrac{1}{2} l_2 l_3 \sin\theta_{23} = \tfrac{1}{2} l_1 l_3 \sin\theta_{13}$$

This gives a face area of 10.1 m².



Priest (1985)

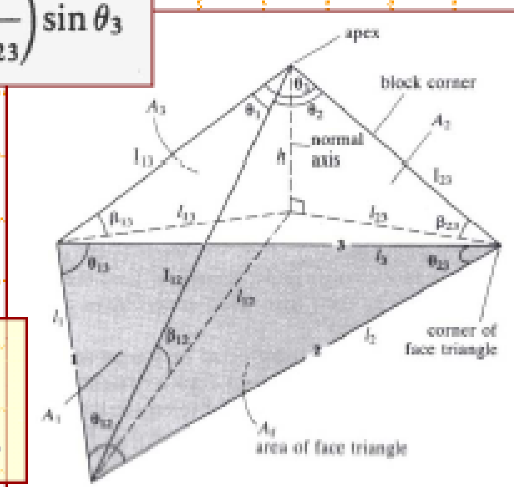
Maximum Wedge Volume

9) The areas of the three internal block surfaces can be found in a similar way from the edge lengths and appropriate internal angles:

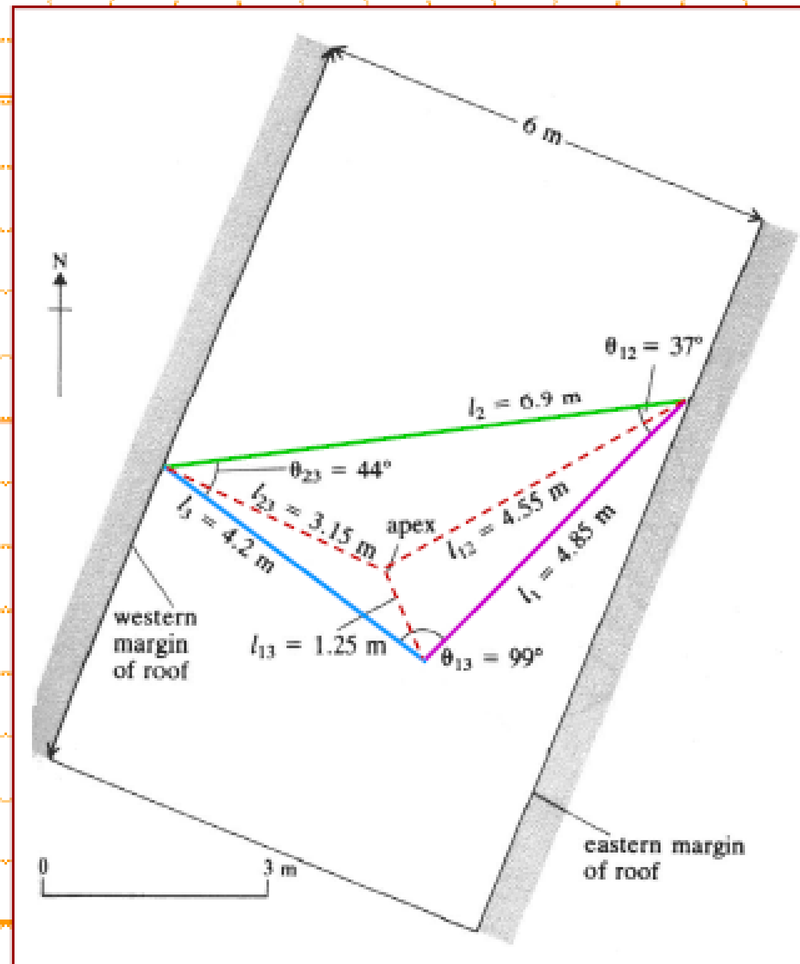
$$A_1 = \frac{1}{2}\left(\frac{l_{12}}{\cos\beta_{12}}\ \frac{l_{13}}{\cos\beta_{13}}\right)\sin\theta_1$$

$$A_2 = \frac{1}{2}\left(\frac{l_{12}}{\cos\beta_{12}}\ \frac{l_{23}}{\cos\beta_{23}}\right)\sin\theta_2$$

$$A_3 = \frac{1}{2}\left(\frac{l_{13}}{\cos\beta_{13}}\ \frac{l_{23}}{\cos\beta_{23}}\right)\sin\theta_3$$



... geometrical properties of a tetrahedral block.



Priest (1985)

Maximum Wedge Volume

10) To find the volume of the wedge, the wedge height and the face area are required. The face area, A_f, has already been found. The wedge height, h, is given by:

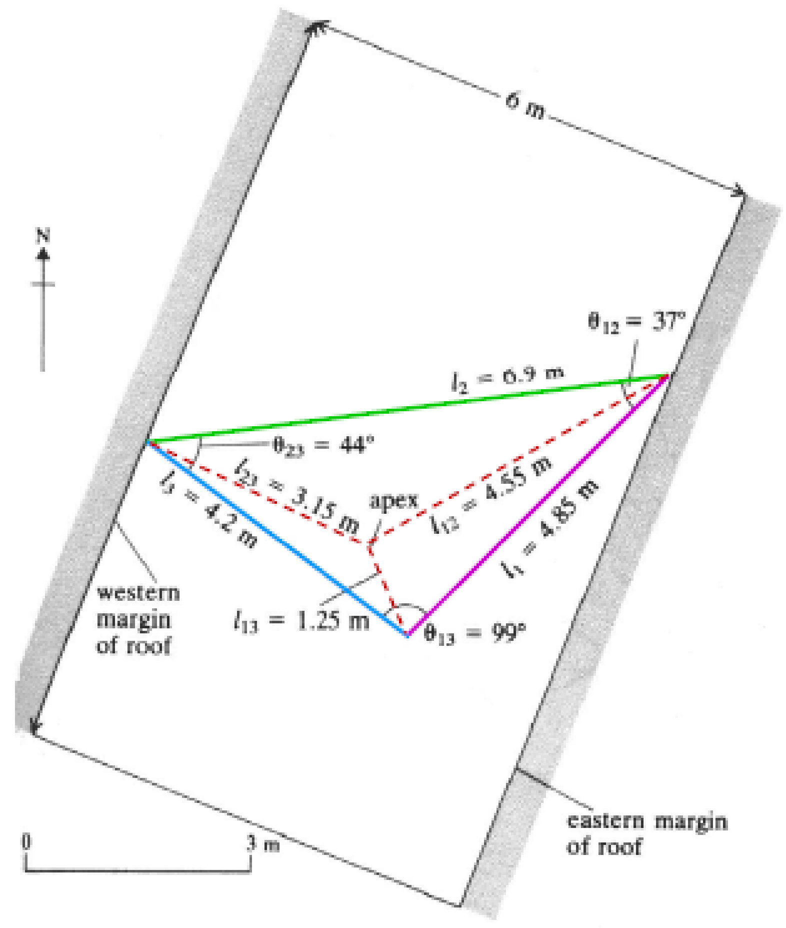
$$h = l_{12} \tan \beta_{12} = l_{23} \tan \beta_{23} = l_{13} \tan \beta_{13}$$

which for this example problem comes to 1.47 m.

The volume, V, of the tetrahedral block is then given as:

$$V = A_f h/3$$

resulting in a block volume of approximately 5 m³.



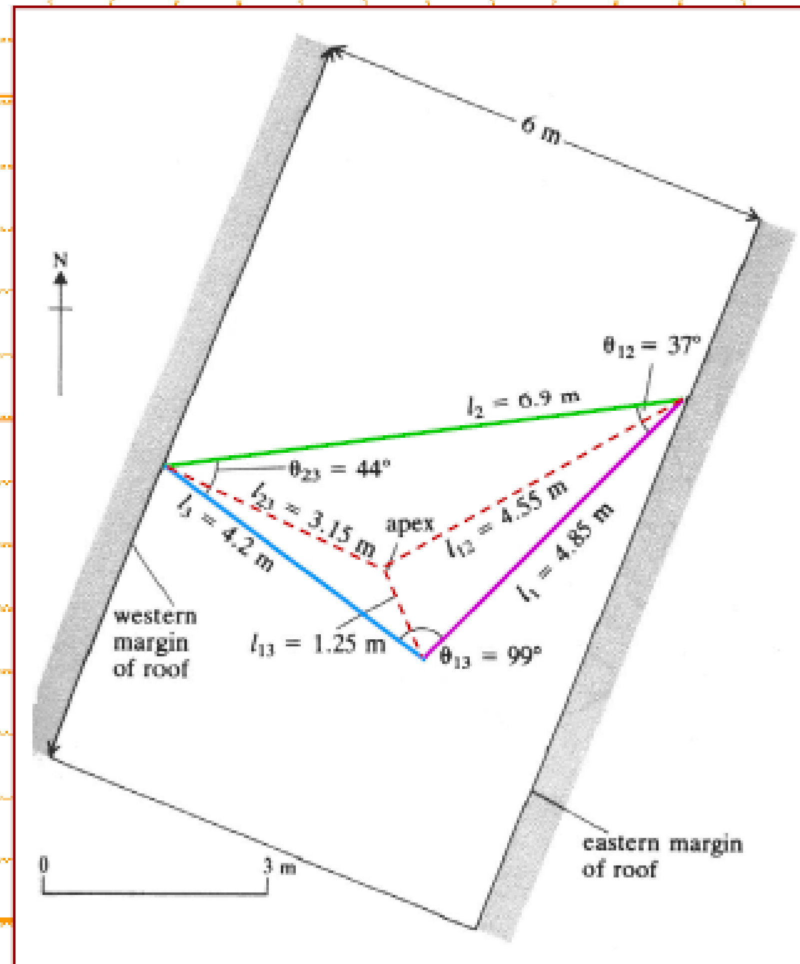
Priest (1985)

Maximum Wedge Volume

11) Now assuming a unit weight of 25 kN/m³ for sedimentary rock, the block would have a weight of approximately 124 kN.

By dividing this value through by the face area, it can be seen that a support pressure of only 12.3 kN/m², distributed over the face triangle, would be required to keep it in place.

This support pressure could, for example, be provided by rock bolts anchored beyond the block at a distance of 2 to 3 m above the excavation roof.



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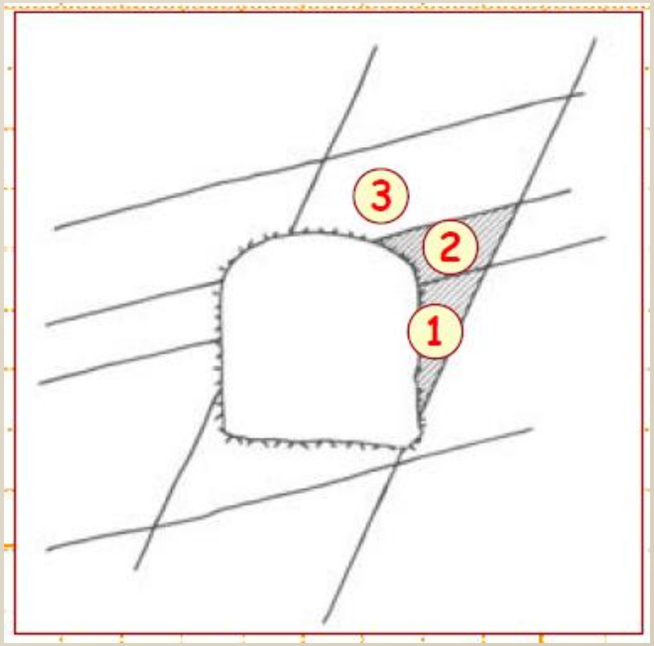
Key Block Analysis

The underlying axiom of block theory is that the failure of an excavation begins at the boundary with the movement of a block into the excavated space. The loss of the first block augments the space, possibly creating an opportunity for the failure of additional blocks, with continuing degradation possibly leading to massive failure.

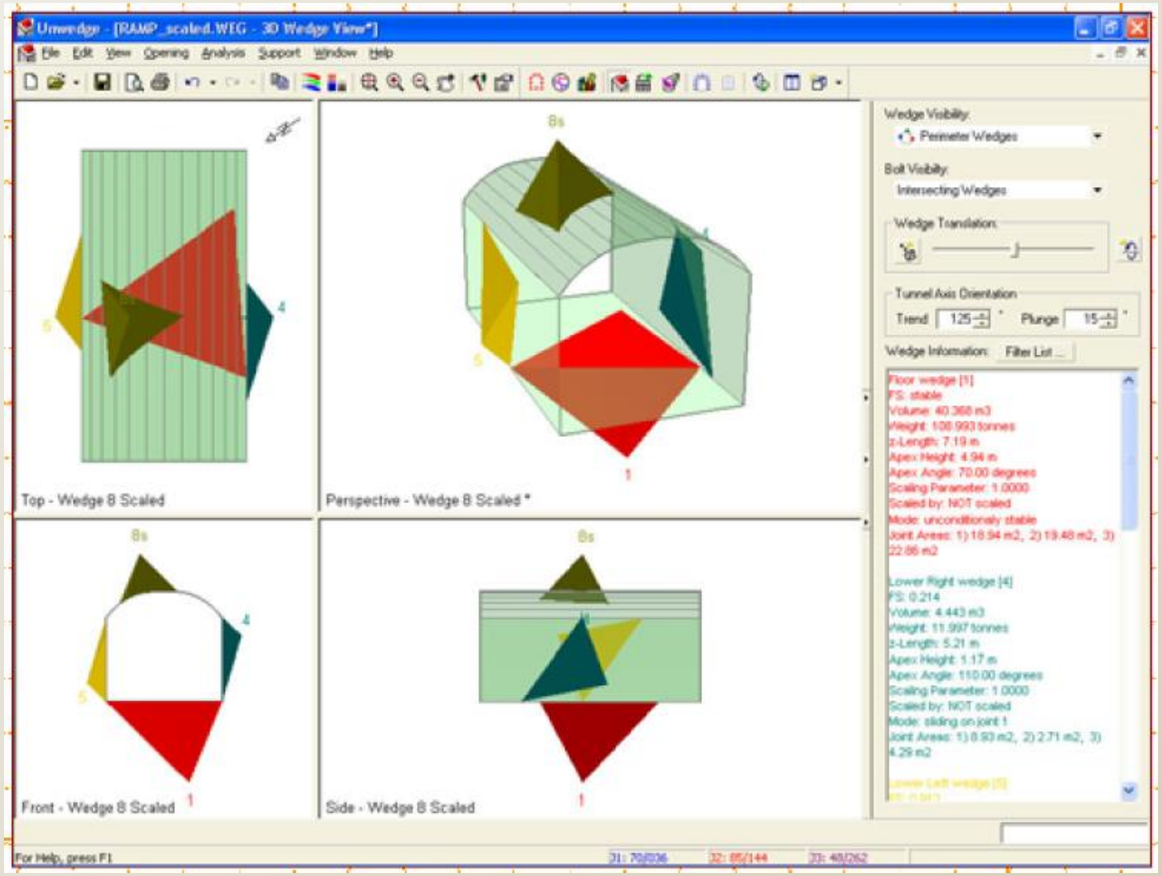
As such, the term key-block identifies any block that would become unstable when intersected by an excavation. The loss of a key-block does not necessarily assure subsequent block failures, but the prevention of its loss does assure stability.



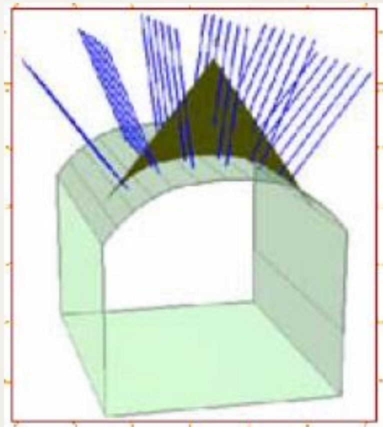
Key-block theory therefore sets out to establish procedures for describing and locating key blocks and for establishing their support requirements.

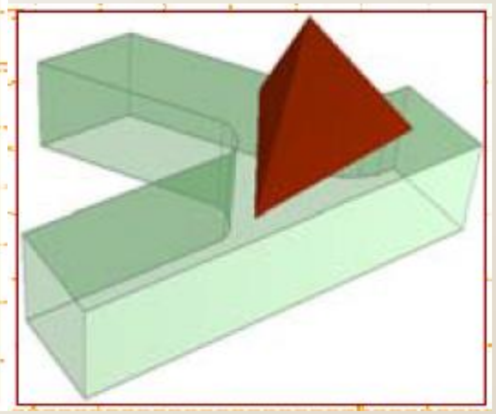
Wedge Analysis – Computer-Aided

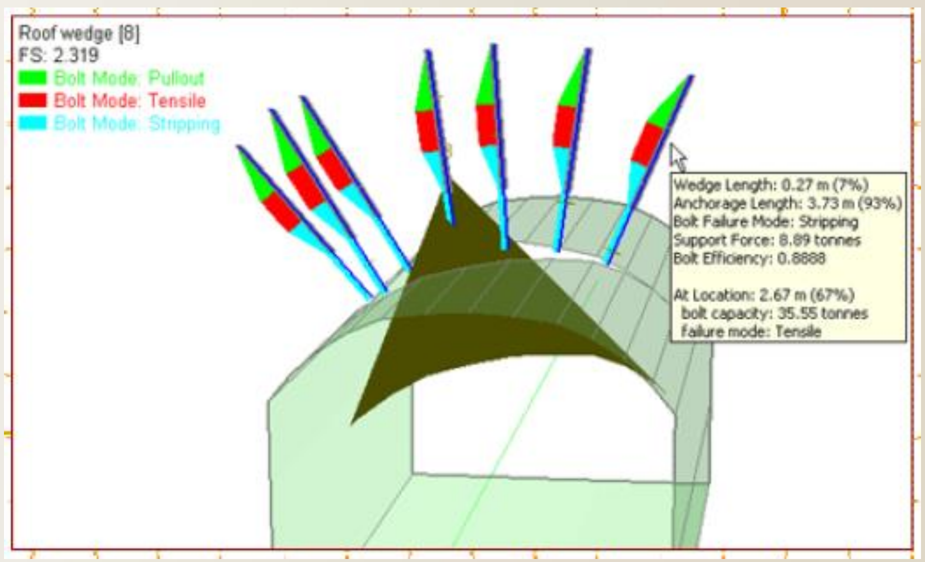
The **3-D** nature of wedge problems (i.e. **size** and **shape** of potential wedges in the rock mass surrounding an opening) necessitates a set of relatively **tedious calculations**. While these can be performed by hand, it is far more efficient to utilise **computer-based techniques**.



Computer-Aided Wedge Analysis in Design



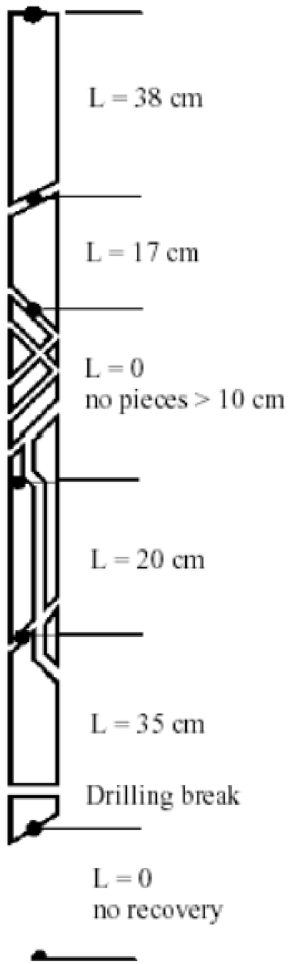




The speed of computer-aided wedge analyses allow them to be employed within the design methodology as a tool directed towards "filter analysis". This is carried out during the preliminary design to determine whether or not there are stability issues for a number of different problem configurations (e.g. a curving tunnel, different drifts in the development of an underground mine, etc.).

LESSON 3. UNDERGROUND EXCAVATION DESIGN CLASSIFICATION

Rock Quality Designation



Total length of core run = 200 cms

$$RQD = \frac{\Sigma \text{ Length of core pieces} > 10 \text{ cm length}}{\text{Total length of core run}} \times 100$$

$$RQD = \frac{38 + 17 + 20 + 35}{200} \times 100 = 55\ \%$$

Measurement and calculation of RQD Rock Quality Designation index (after Deere 1989)

Rock Structure Rating (RSR)

- Wickham et al (1972) described a quantitative method for describing the quality of rock mass and for selecting appropriate support on the basis of their Rock Structure Rating (RSR) classification.
- The system is historical (developed using relatively small tunnels but it was the first method which made references to shotcrete support).
- The significance of the RSR system:

  Introducing the concept of rating each of the components listed below to arrive at a numerical value

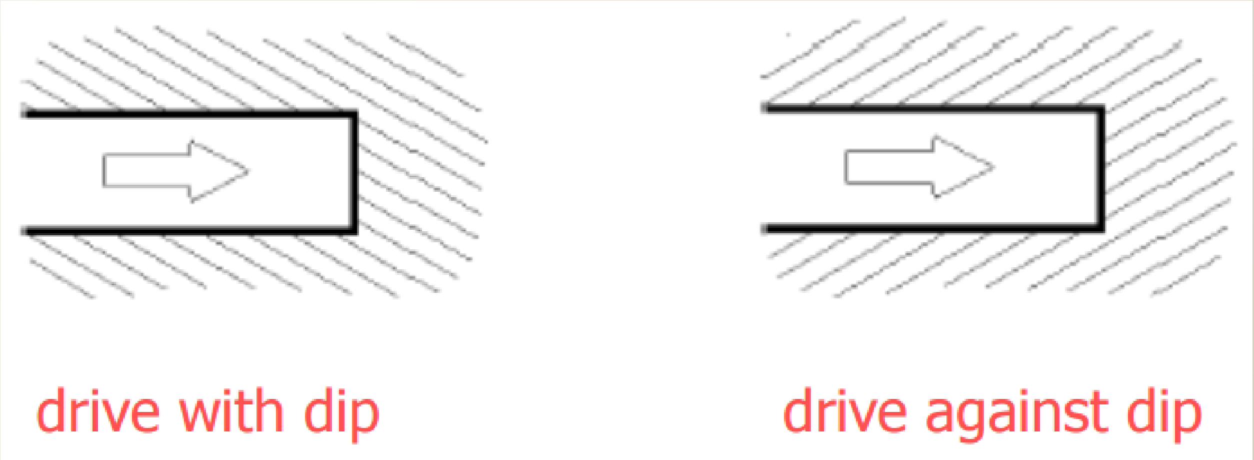
$$RSR = A + B + C$$

Rock Structure Rating (RSR)

1. Parameter A, Geology: General appraisal of geological structure on the basis of:
   a. Rock type origin (igneous, metamorphic, sedimentary)
   b. Rock hardness (hard, medium, soft, decomposed)
   c. Geological structure (massive, slightly, faulted/folded, moderately faulted/folded, intensely faulted/folded)

Rock Structure Rating (RSR)

2. Parameter B, Geometry: Effect of discontinuity pattern with respect to the direction of the tunnel drive on the basis of:
   a. Joint spacing
   b. Joint orientation (strike and dip)
   c. Direction of tunnel drive



Rock Structure Rating (RSR)

3. Parameter C, Effect of Groundwater inflow and joint condition on the basis of:
   a. Overall rock mass quality on the basis of A and B combined
   b. Joint condition (good, fair, poor)
   c. Amount of water inflow (in gallons per minute per 1000 feet of tunnel)

Maximum RSR = 100

	Basic Rock Type				Geological Structure			
	Hard	Medium	Soft	Decomposed				
Igneous	1	2	3	4		Slightly	Moderately	Intensively
Metamorphic	1	2	3	4		Folded or	Folded or	Folded or
Sedimentary	2	3	4	4	Massive	Faulted	Faulted	Faulted
Type 1					30	22	15	9
Type 2					27	20	13	8
Type 3					24	18	12	7
Type 4					19	15	10	6

For example, a hard metamorphic rock which is slightly folded or faulted has a rating of A = 22.

	Strike ⊥ to Axis					Strike ‖ to Axis		
	Direction of Drive					Direction of Drive		
	Both	With Dip		Against Dip		Either direction		
	Dip of Prominent Joints [a]					Dip of Prominent Joints		
Average joint spacing	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1. Very closely jointed, < 2 in	9	11	13	10	12	9	9	7
2. Closely jointed, 2-6 in	13	16	19	15	17	14	14	11
3. Moderately jointed, 6-12 in	23	24	28	19	22	23	23	19
4. Moderate to blocky, 1-2 ft	30	32	36	25	28	30	28	24
5. Blocky to massive, 2-4 ft	36	38	40	33	35	36	24	28
6. Massive, > 4 ft	40	43	45	37	40	40	38	34

The rock mass is moderately jointed, with joints striking perpendicular to the tunnel axis which is being driven eastwest, are dipping at between 20° and 50°. The rating for B = 24 driving with dip.

Anticipated water inflow gpm/1000 ft of tunnel	Sum of Parameters *A* + *B*: 13 - 44			Sum of Parameters *A* + *B*: 45 - 75		
	Joint Condition [b]					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight, < 200 gpm	19	15	9	23	19	14
Moderate, 200-1000 gpm	15	22	7	21	16	12
Heavy, > 1000 gp	10	8	6	18	14	10

[a] Dip: flat: 0-20°; dipping: 20-50°; and vertical: 50-90°

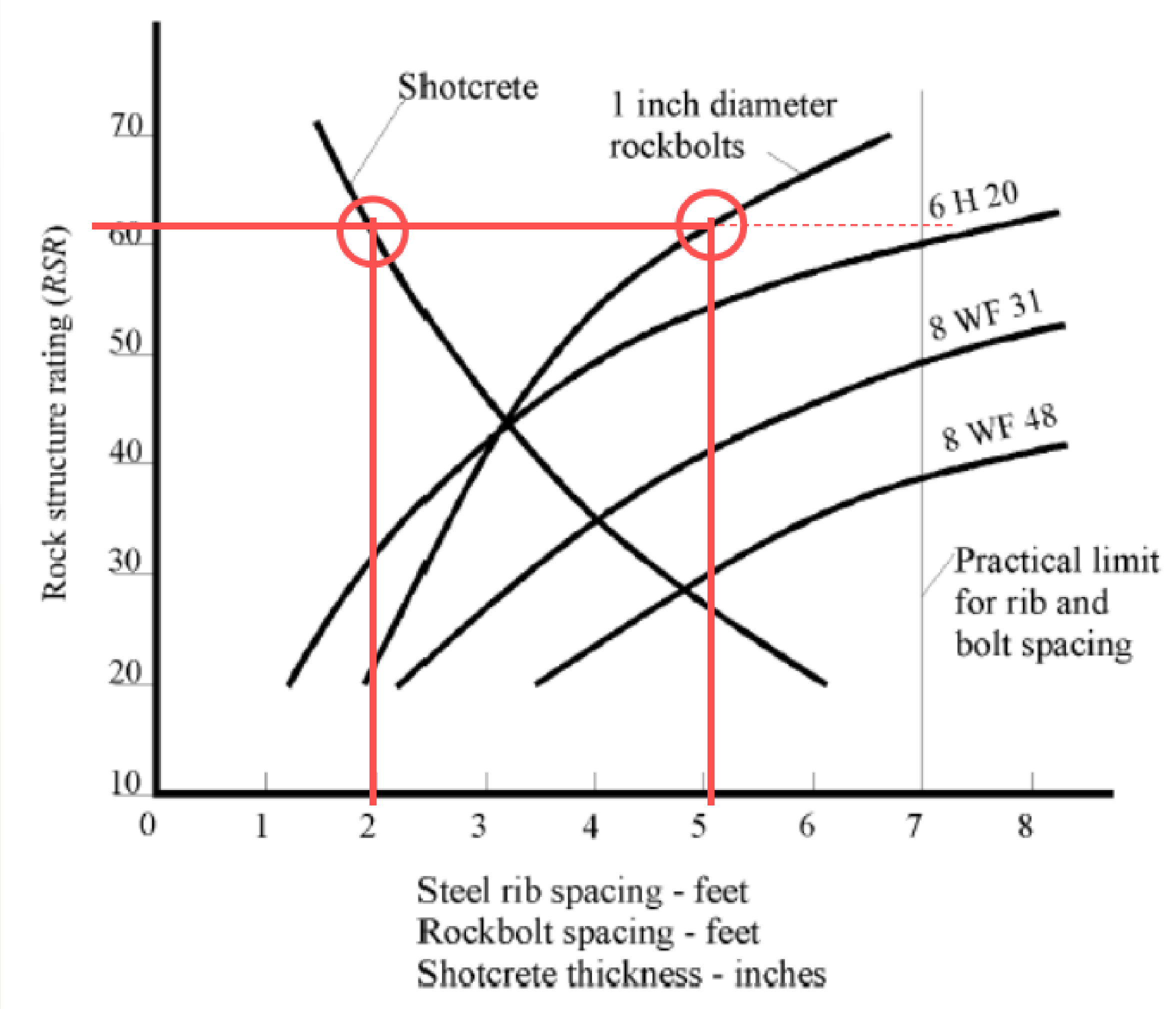
[b] Joint condition: good = tight or cemented; fair = slightly weathered or altered; poor = severely weathered, altered or open

The value of A + B = 46 and this means that, for joints of fair condition (slightly weathered and altered) and a moderate water inflow of between 200 and 1000 gallons per minute.

The rating gives C = 16.

Hence, the final value of the rock structure rating RSR = A + B + C = 62.

RSR Parameter C: Groundwater, joint condition



RSR support estimates for a 24 ft (7.3m) diameter circular tunnel. Note that rockbolts and shotcrete are generally used together (after Wickham et al. 1972).

Rock Mass Rating (RMR)

Rock Mass Rating (RMR) after Bieniawski (1976). It is also called Geomechanics Classification. Modifications were made. This is the 1989 version 6 parameters are used to classify the rock mass:

1. Uniaxial compressive strength of rock material
2. Rock Quality Designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Groundwater conditions
6. Orientation of discontinuities

Rock Mass Rating (RMR)

Tunnel through a slightly weathered granite with a dominant joint set dipping against the direction of the drive.

Point-load strength: 8 MPa

RQD: 70%

The slightly rough and slightly weathered joints with separation of < 1 mm, are spaced at 300mm. Tunnelling conditions are anticipated to be wet.

Rock Mass Rating (RMR)

The RMR value is determined as follows:

Item	*Value*	*Rating*
Point load index	8 MPa	12
RQD	70%	13
Spacing of discontinuities	300 mm	10
Condition of discontinuities	Note 1	22
Groundwater	Wet	7
Adjustment for joint orientation	Note 2	-5
	Total	59

Rock Mass Rating (RMR)

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

Rock Mass Rating (RMR)

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**			
Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
$\boxed{\text{Fair}}$	Unfavourable	Fair	

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Rock Mass Rating (RMR)

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)						
Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS						
Rating		100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number		I	II	III	IV	V
Description		Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
D. MEANING OF ROCK CLASSES						
Class number		I	II	III	IV	V
Average stand-up time		20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)		> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)		> 45	35 - 45	25 - 35	15 - 25	< 15
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions						
Discontinuity length (persistence) Rating		< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating		None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating		Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating		None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings		Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

Rock Mass Rating (RMR)

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock *RMR*: 81-100	Full face, 3 m advance.	Generally no support required except spot bolting.		
II - Good rock *RMR*: 61-80	Full face , 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock *RMR*: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None.

Rock Mass Rating (RMR)

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
IV - Poor rock *RMR*: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor rock *RMR*: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Rock Tunnelling Quality Index, Q

The system was proposed by Barton et al. (1974) at the Norwegian Geotechnical Institute in order to determine the rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1000 and is defined by

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

RQD	is the Rock Quality Designation
J_n	is the joint set number
J_r	is the joint roughness number
J_a	is the joint alteration number
J_w	is the joint water reduction factor
SRF	is the stress reduction factor

Rock Tunnelling Quality Index, Q

The first quotient (RQD/Jn), representing the structure of the rock mass, is a crude measure of the block of particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400.

The second quotient (Jr/Ja) represents the roughness and frictional characteristics of the joint walls or filling materials.

The third quotient (Jw/SRF) consists of two stress parameters, SFR is a measure of: 1) loosening load in the case of an excavation through shear zones and clay bearing rock, 2) rock stress in competent rock, and 3) squeezing loads in plastic incompetent rock. It can be seen as a total stress parameter.

The parameter Jw is a measure of water pressure, which has an adverse effect of the shear strength of joints due to a reduction in effective normal stress. The quotient (Jw/SRF) is a complicated emperical factor describing the “active stress”.

Rock Tunnelling Quality Index, Q

It appears that the rock tunnelling quality Q can now be considered to be a function of only three parameters which are crude measures of:

1. Block size	(RQD/J_n)
2. Inter-block shear strength	(J_r/J_a)
3. Active stress	(J_w/SRF)

Undoubtedly, there are several parameters which could be added to improve the accuracy of the classification system: One of these be the joint orientation Structural orientation in relation to excavation axis.

Rock Tunnelling Quality Index, Q

Jn, Jr, Ja appear to play a more important role than orientation, because the number of joint sets determine the degree of freedom for block movement, and the friction and dilatation characteristics can vary more than the down-dip gravitational component of unfavourable oriented joints.

If joint orientation had been included the classification would have been less general, and its essential simplicity lost.

Rock Tunnelling Quality Index, Q

DESCRIPTION	VALUE	NOTES
1. ROCK QUALITY DESIGNATION	*RQD*	
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as ≤ 10 (including 0),
B. Poor	25 - 50	a nominal value of 10 is used to evaluate *Q*.
C. Fair	50 - 75	
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently
E. Excellent	90 - 100	accurate.
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use $(3.0 \times J_n)$
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use $(2.0 \times J_n)$
J. Crushed rock, earthlike	20	

Rock Tunnelling Quality Index, Q

3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact		
b. Rock wall contact before 10 cm shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is
E. Rough or irregular, planar	1.5	greater than 3 m.
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having
c. No rock wall contact when sheared		lineations, provided that the lineations are oriented for
H. Zones containing clay minerals thick	1.0	minimum strength.
enough to prevent rock wall contact	(nominal)	
J. Sandy, gravely or crushed zone thick	1.0	
enough to prevent rock wall contact	(nominal)	

Rock Tunnelling Quality Index, Q

4. JOINT ALTERATION NUMBER *a. Rock wall contact*	J_a	ϕr degrees (approx.)	
A. Tightly healed, hard, non-softening, impermeable filling	0.75		1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25 - 35	
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30	
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25	
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)	4.0	8 - 16	

Rock Tunnelling Quality Index, Q

DESCRIPTION	VALUE	NOTES
4, JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)
b. Rock wall contact before 10 cm shear		
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16
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	6 - 12
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	6 - 24
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0	
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	

Rock Tunnelling Quality Index, Q

5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	

Rock Tunnelling Quality Index, Q

6. STRESS REDUCTION FACTOR	SRF	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated		
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)	10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)	5.0	
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)	2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)	5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)	2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)	5.0	

Rock Tunnelling Quality Index, Q

DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			*SRF*	
b. Competent rock, rock stress problems				
	σ_c/σ_1	σ_t/σ_1		
H. Low stress, near surface	> 200	> 13	2.5	
J. Medium stress	200 - 10	13 - 0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. Mild squeezing rock pressure			5 - 10	
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

2. For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.

3. Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H).

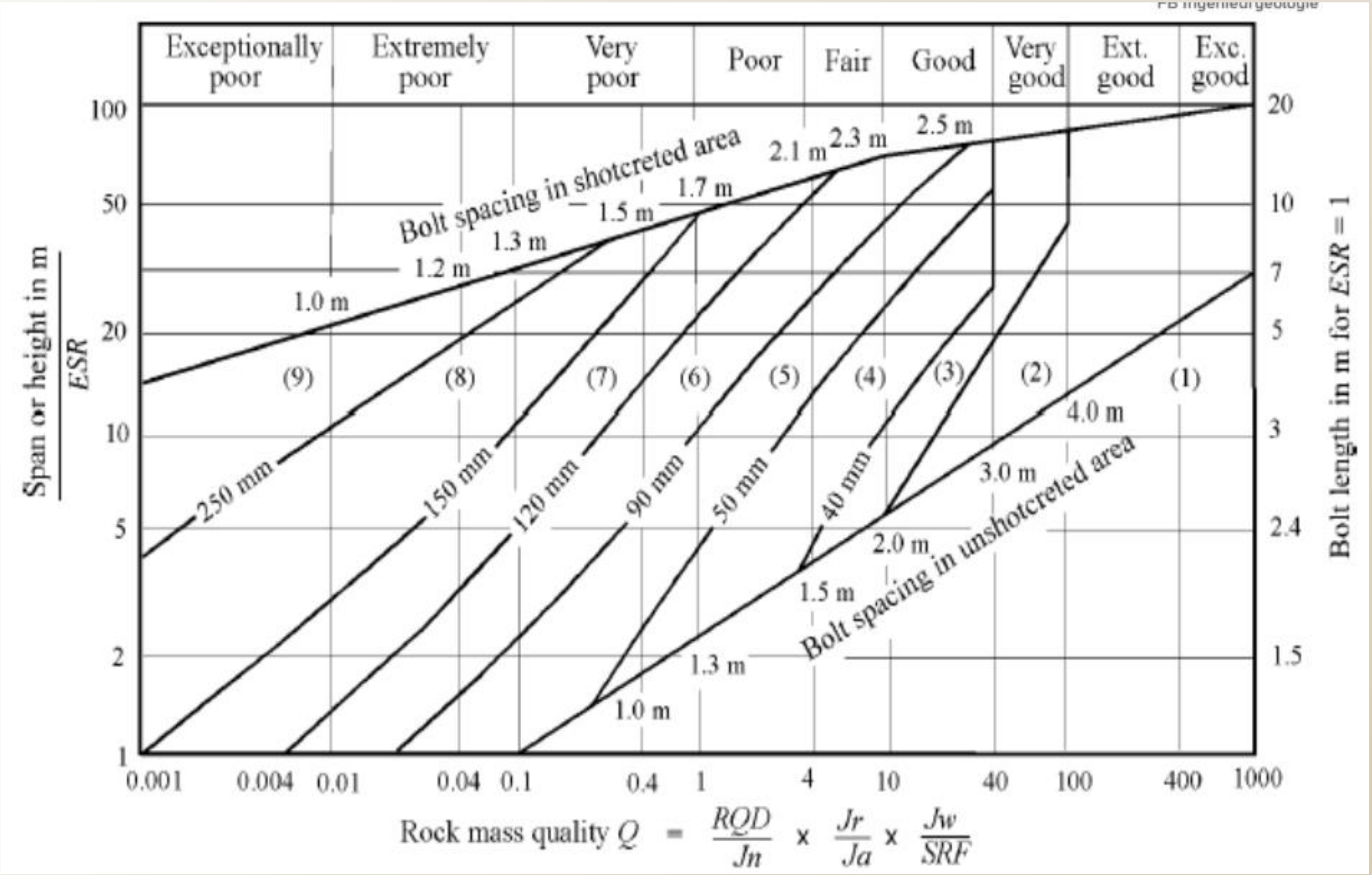
Rock Tunnelling Quality Index, Q

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass Quality (*Q*), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3\ J_v$ (approx.), where J_v = total number of joints per m^3 ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating *Q*. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Rock Tunnelling Quality Index, Q



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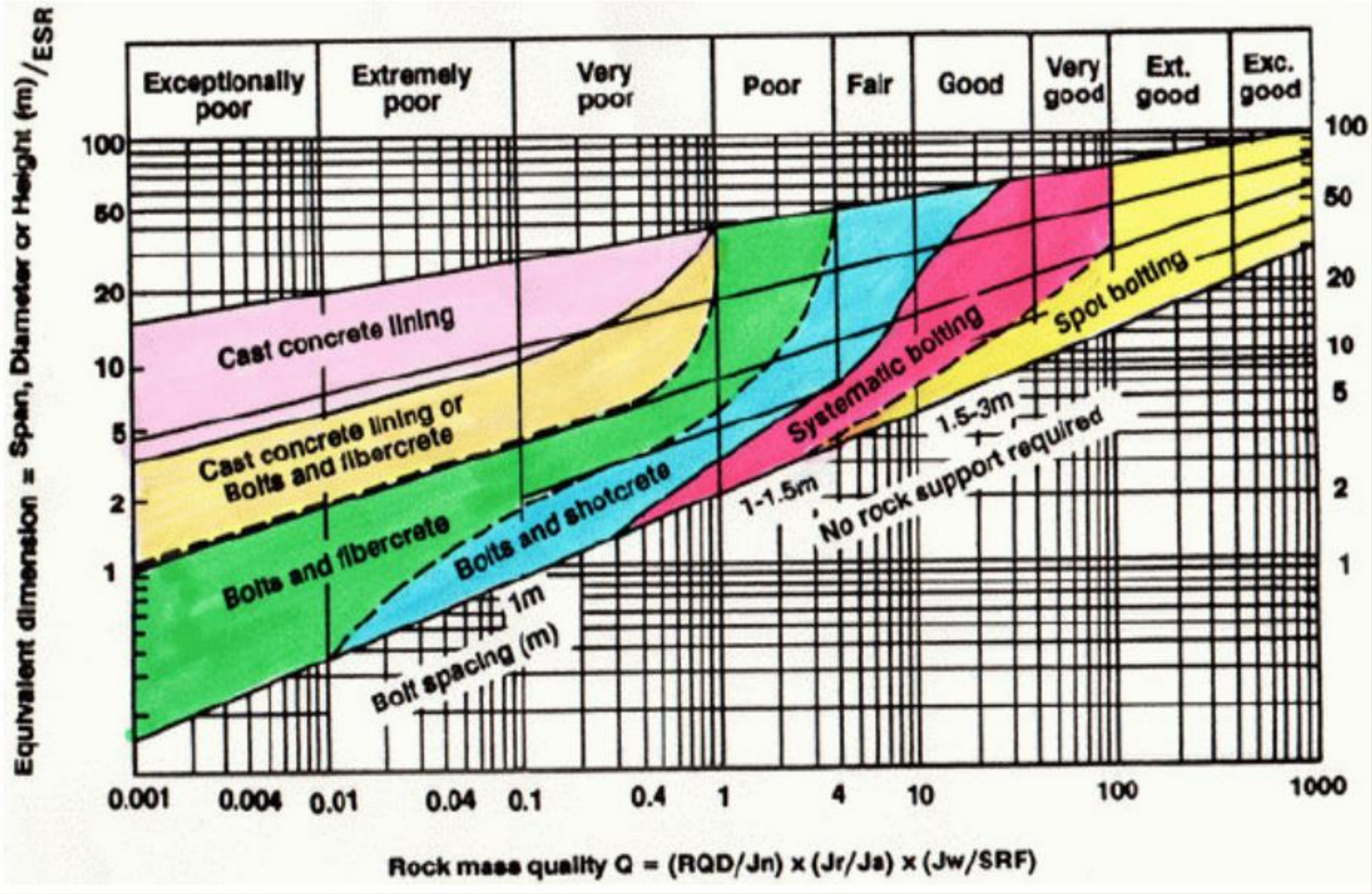
Rock Tunnelling Quality Index, Q

REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40-100 mm unreinforced shotcrete
5) Fibre reinforced shotcrete, 50 - 90 mm, and bolting
6) Fibre reinforced shotcrete, 90 - 120 mm, and bolting
7) Fibre reinforced shotcrete, 120 - 150 mm, and bolting
8) Fibre reinforced shotcrete, > 150 mm, with reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Rock Tunnelling Quality Index, Q



Rock Tunnelling Quality Index, Q

15 m span underground mine

Granite Depth: 2100m

Tow sets of joints RQD: 85% - 95%

Uniaxial compressive strength: 170MPa

Magnitude of the horizontal stress to vertical stress: 1.5

No water flowing

$J_n = 4$

$J_r = 3$

$J_a = 1.0$

$J_w = 1.0$

Depth = 2100m => vertical stress = 57MPa

Rock Tunnelling Quality Index, Q

Major principle stress = 85 MPa

Uniaxial compressive strength = 170MPa

Ratio sigmac/sigma1=2 => heavy rock bursts expected

SRF between 10 and 20 => assumed SRF = 15

$$Q = \frac{90}{4} \times \frac{3}{1} \times \frac{1}{15} = 4.5$$

Rock Tunnelling Quality Index, Q

In relating the value of the index Q to the stability and support requirements of underground excavations, Barton et al. (1974) defined an additional parameter which they called the Equivalent Dimension, De, of the excavation. This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio, ESR. Hence:

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio } ESR}$$

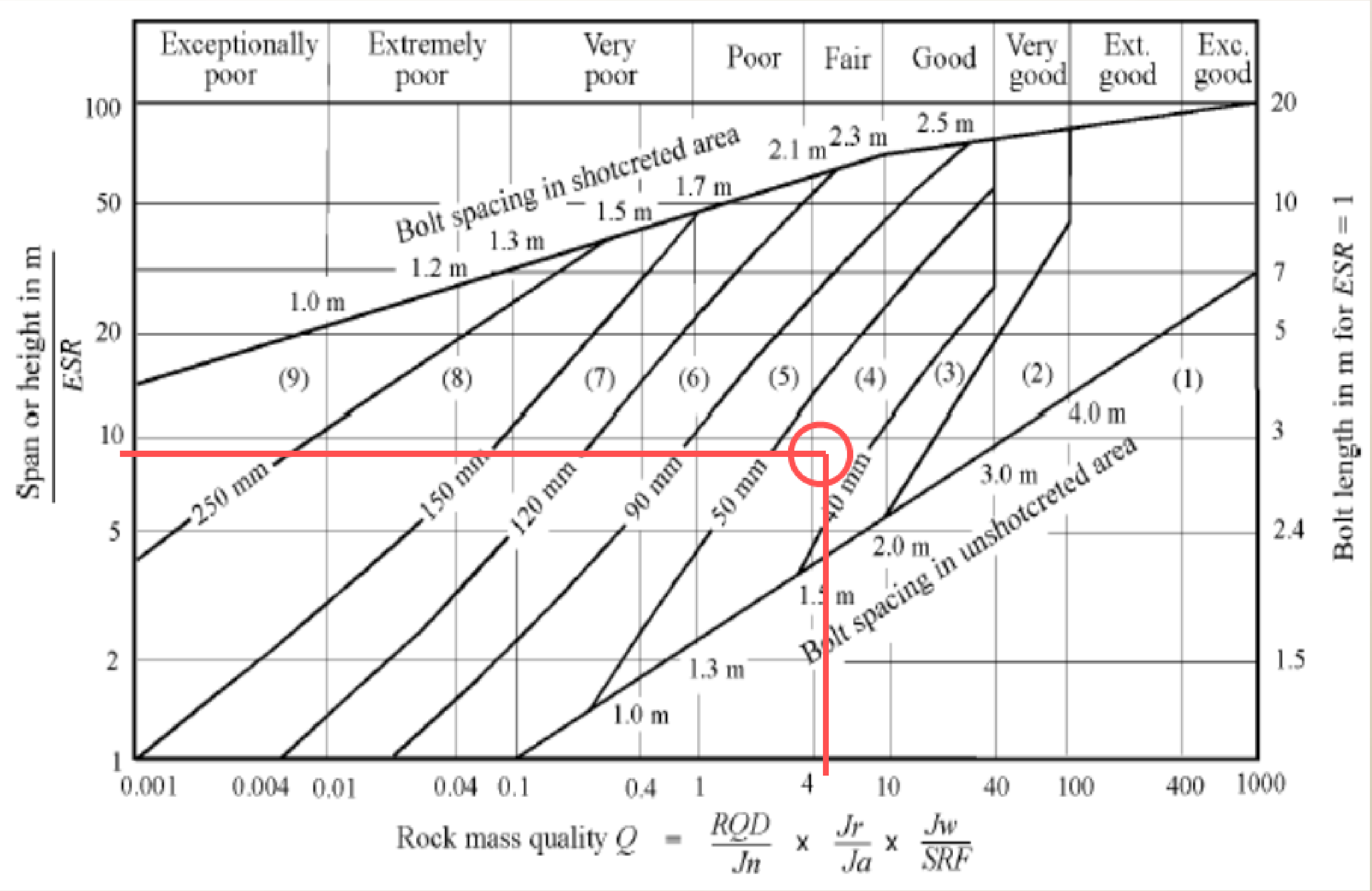
Rock Tunnelling Quality Index, Q

Excavation category		*ESR*
A	Temporary mine openings.	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Rock Tunnelling Quality Index, Q

- Permanent mining ESR = 1.6
- De = 15/1.6 = 9.4
- Q = 4.5

Rock Tunnelling Quality Index, Q



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Rock Tunnelling Quality Index, Q

Barton et al. provided also rock bolt length:

B = excavation width B

L = Length of the rock bolt

$$L = \frac{2 + 0.15B}{ESR}$$

Maximum unsupported span:

$$\text{Maximum span (unsupported)} = 2\,ESR\,Q^{0.4}$$

Rock Tunnelling Quality Index, Q

Guideline for the Geotechnical Design of Underground Structures with Conventional Excavation

by Austrian Society of Rock Mechanics

Phase 1: Design

GroundTypes

Ground Behaviour Types

System Behaviour Types (SB)

- Geotechnical Report

Phase 2: Construction

Evaluation of the predicted SB

Ground Type: Ground with similar properties

Ground Behaviour: Reaction of the ground to the excavation of the full profile without consideration of sequential excavation and support

Behaviour Type: General categories describing similar Ground Behaviours with respect to failure modes and displacement characteristics

Estimation of in situ deformation modulus

- The in situ deformation modulus of a rock mass is an important parameter in any form of numerical analysis and in the interpretation of monitored deformation around underground openings.
- Since this parameter is very difficult and expensive to determine in the field, several attempts have been made to develop methods for estimating its value, based upon rock mass classifications.

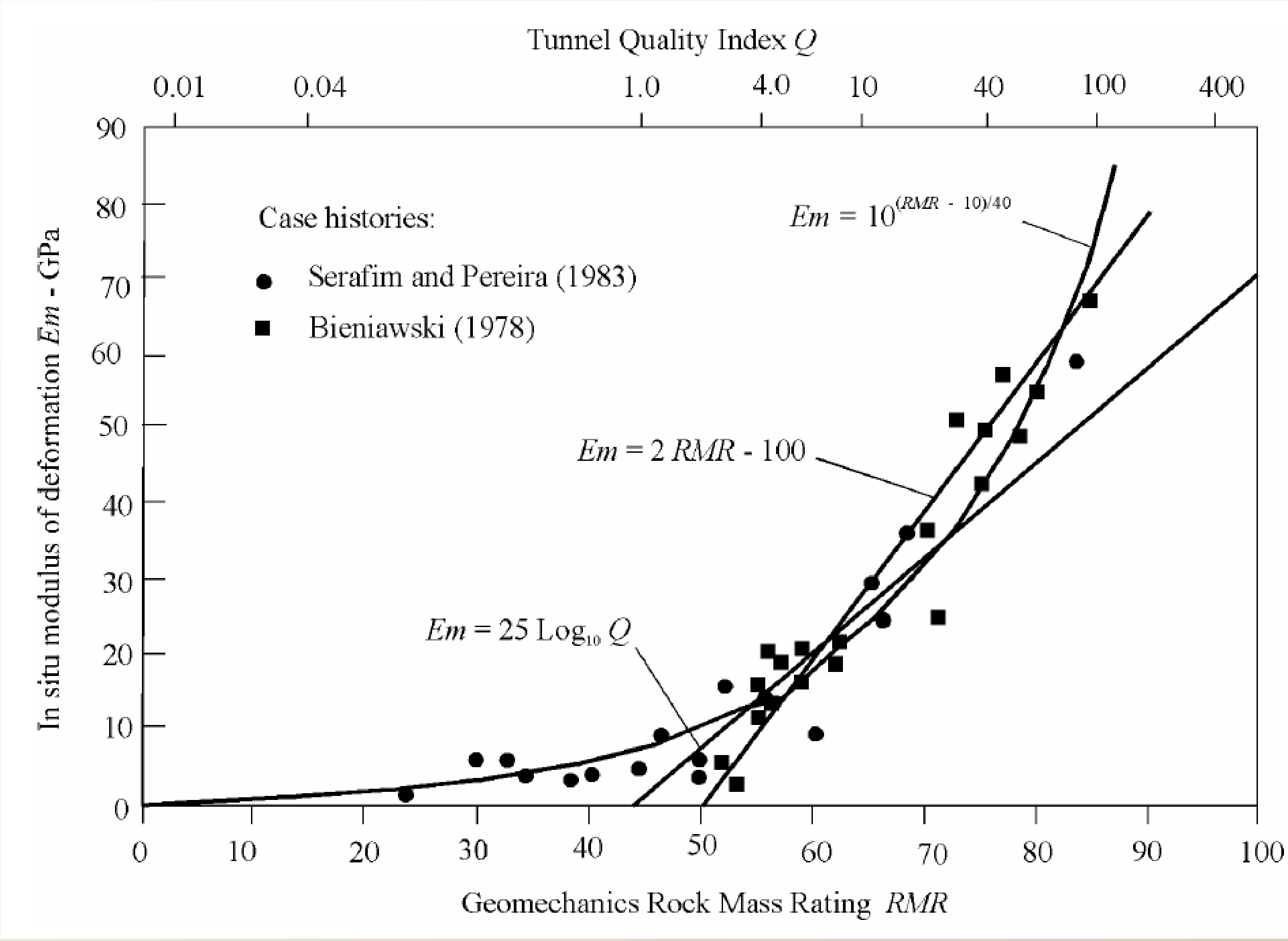
Estimation of in situ deformation modulus

- In the 1960s several attempts were made to use Deere's *RQD* for estimating in situ deformation modulus, but this approach is seldom used today (Deere and Deere, 1988).
- Bieniawski (1978) analysed a number of case histories and proposed the following relationship for estimating the in situ deformation modulus, *Em*, from *RMR*:

$$E_m = 2RMR - 100$$

Estimation of in situ deformation modulus

Serafim and Pereira (1983) proposed the following relationship between *Em* and *RMR*:

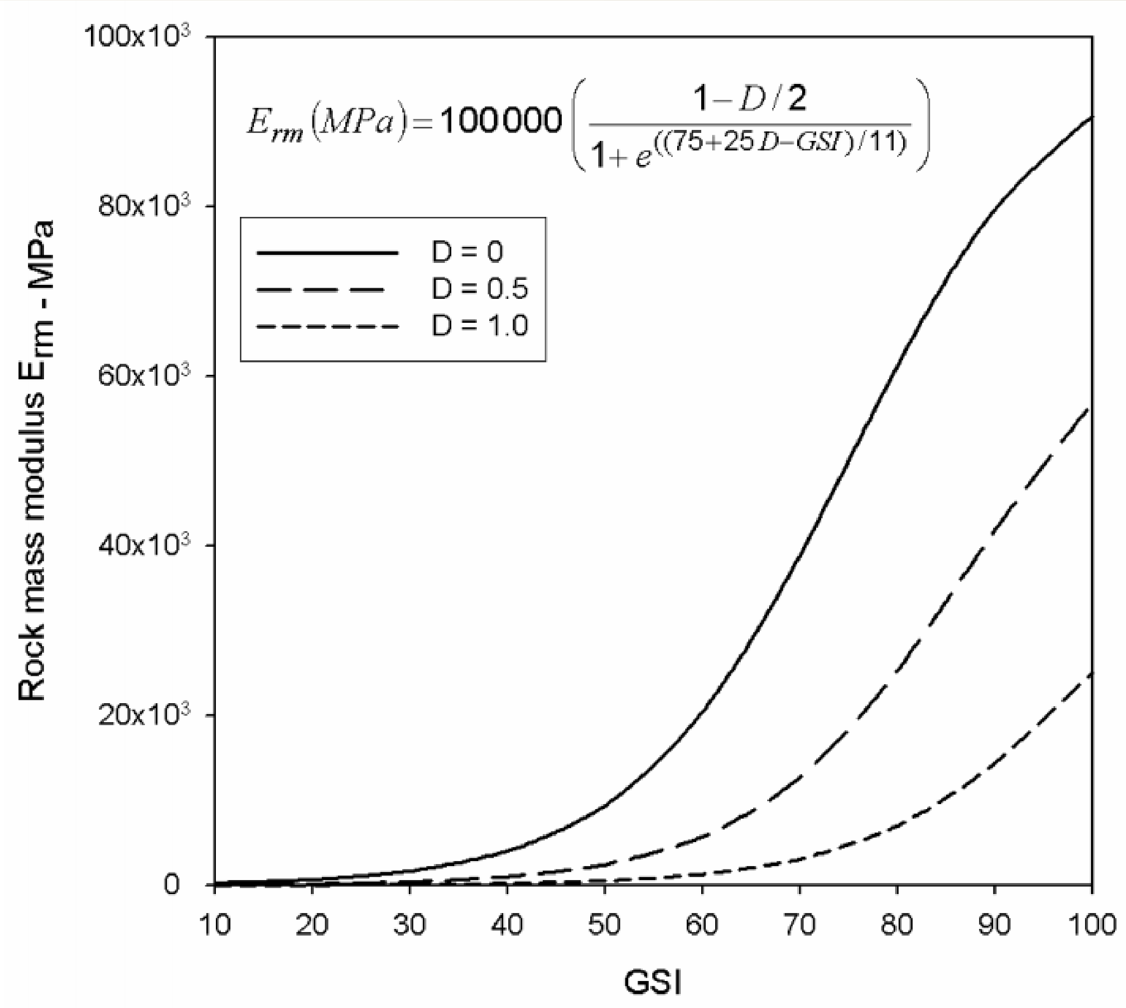


Estimation of in situ deformation modulus

Based on an analysis of an extensive data set from China and Taiwan, Hoek et al (2006) have proposed two new equations for empirical estimates of the deformation modulus of rock masses.

These estimates are based on the Geological Strength Index (GSI), the deformation modulus of the intact rock (Ei) and the rock mass damage factor (D).

Estimation of in situ deformation modulus



(Hoek-Diederichs)

LESSON 4. TUNNELLING METHOD AND ROCK PRESSURE

TUNNELLING METHOD WITH TUNNEL CONSTRUCTION USING DRILL-AND-BLAST

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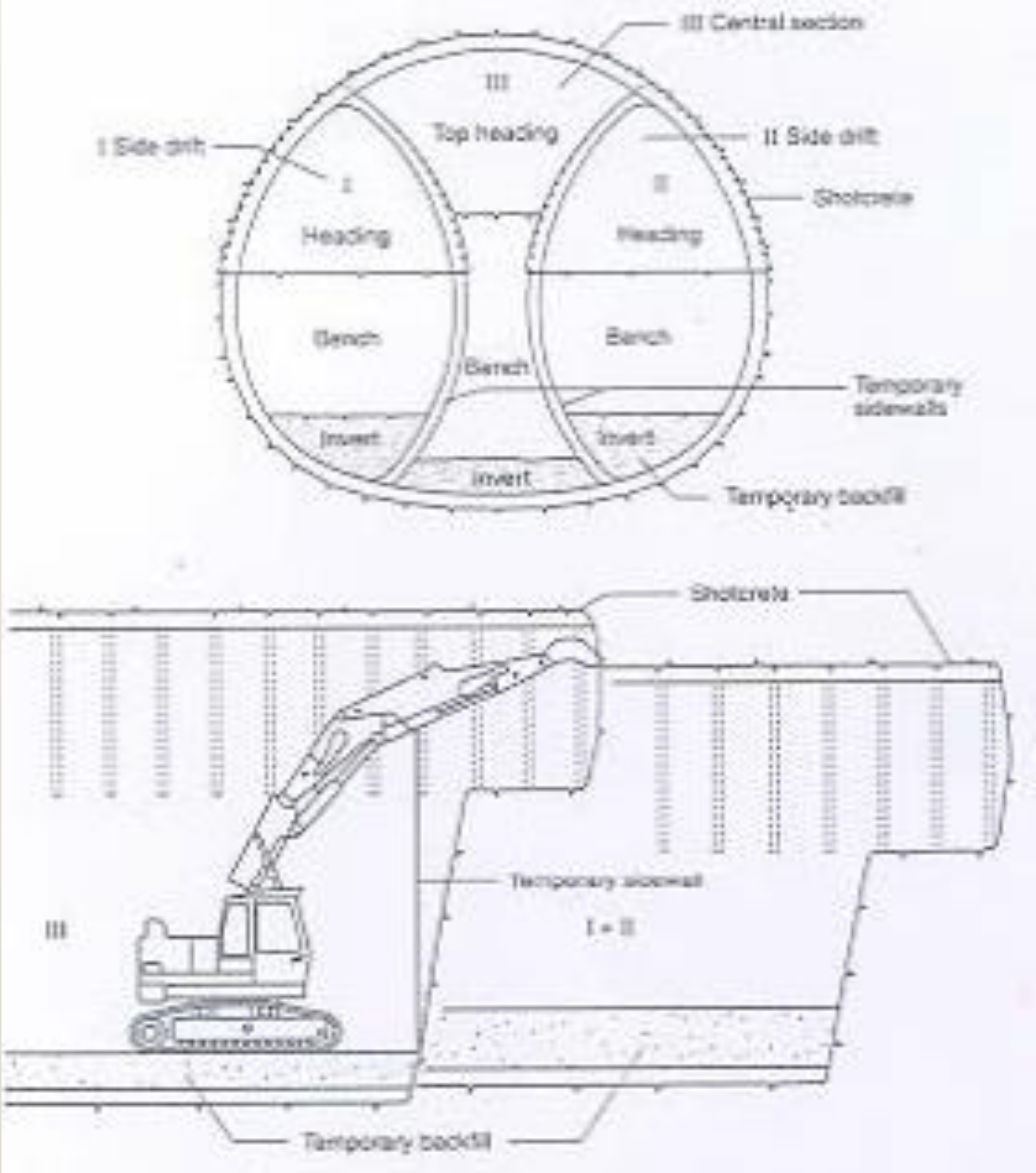
Entrance arrangement to the tunnel portal for the 3.7km long Tai Lam Tunnel of Route 3 at Kam Tin as seen in early 1996. Tai Lam Tunnel consists two main tunnels, each measures 15.5m wide and 10.5m high, and a 950m long servicing duct for ventilation and other supply purposes in between the main tunnels

The erection of the gantry-type formwork for the forming of tunnel lining at entrance of the tunnel portal on Ting Kau side.

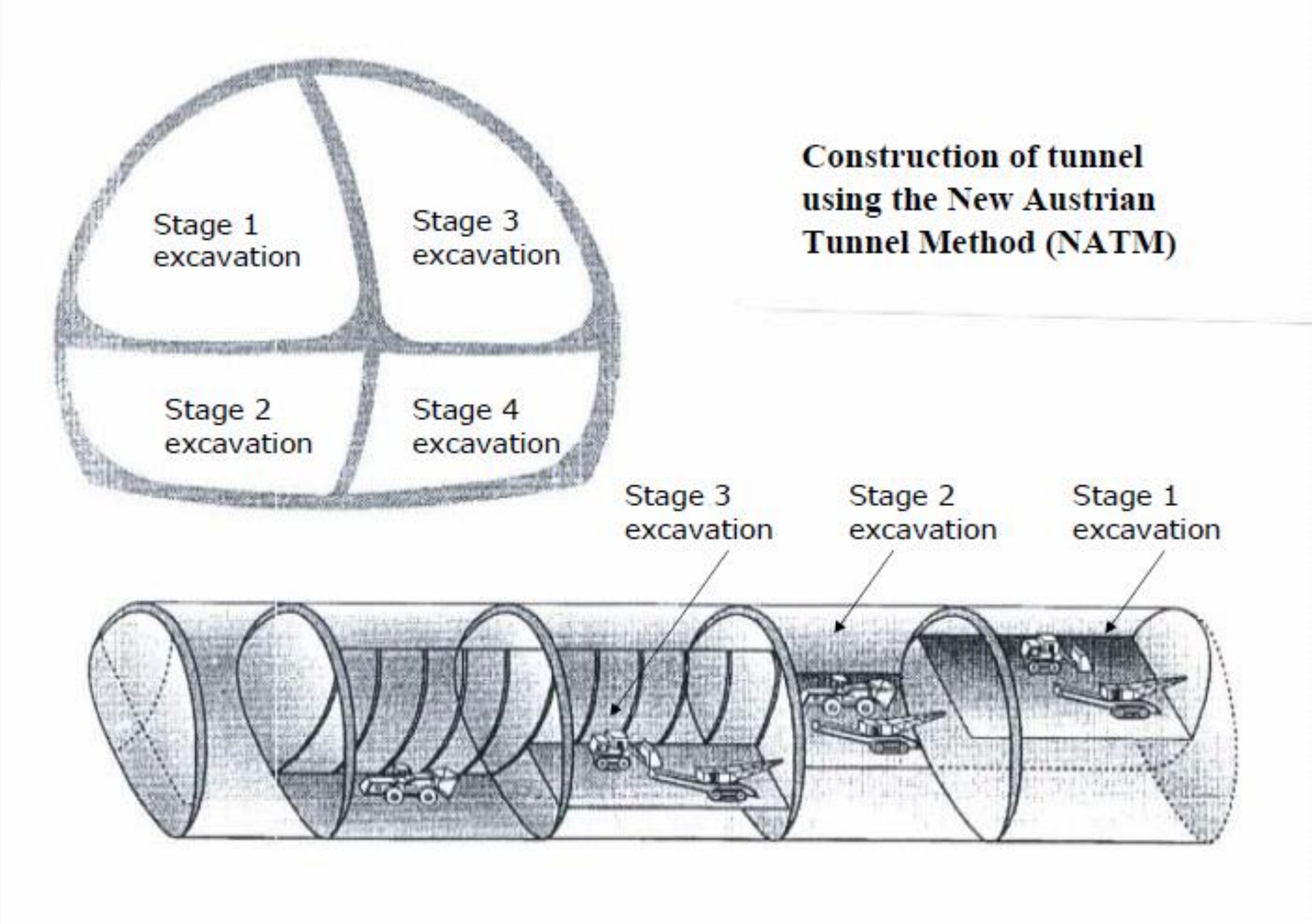
A trial tunnel section being formed

A trial section of tunnel excavation making use of the concept of New Austrian Tunnel Method

Drawing showing the excavation sequences sing the New Austrian Tunnel Method. The principle of constructing large sectioned tunnel using this method is to subdivide the tunnel section into several arched smaller sections for the sake of easier control and safer supporting during excavation. The newly formed surfaces are often required to temporary supported by girder sections, shotcrete, nails or anchors.



Construction of tunnel using the New Austrian Tunnel Method (NATM)



After the tunnel formed by drill and blast process, the newly formed tunnel surface is to be lined with an in-situ concrete lining to stabilize the exposed soil or rock faces. The photo shows the gantry-type formwork used to form the in-situ concrete lining.

Interior of the Tai Lam Tunnel before paving. The installation of the service void by the use of precast ceiling panel is also taking place

The machine, known as Jumbo tunneling machine, is used to drill and form holes inside the tunnel for the placing of explosive to activate the blasting. This machine is computer controlled and can drill 3 holes at the same time with direction or angle precisely set.

TUNNELLING METHOD WITH TUNNEL CONSTRUCTION USING TUNNEL BORING MACHINE (TBM)

The tunnel boring machine for the forming of the 3.8m diameter tunnel tube on Butterfly Valley side. The tunnel boring machine for the forming of the 3.8m diameter tunnel tube on Butterfly Valley side.

Close up of the cutter head. The cutting disc can cut into hard rock and the granulated spoils will be collected and removed by a conveyor system that is positioned immediate at the back of the cutting head

Soil disposal wagons at the disposal area on Butterfly Valley side portal. The spoil will be kept at this location waiting for the removal off-site by dumping vehicles

Arrangement of the portal as viewed from the tunnel exit. The spoil disposal area is located on the right side of the exit with rail track heading to that direction. Rail track on the left is the depot and servicing centre for the soil disposal wagons, as well as for general loading and unloading purposes

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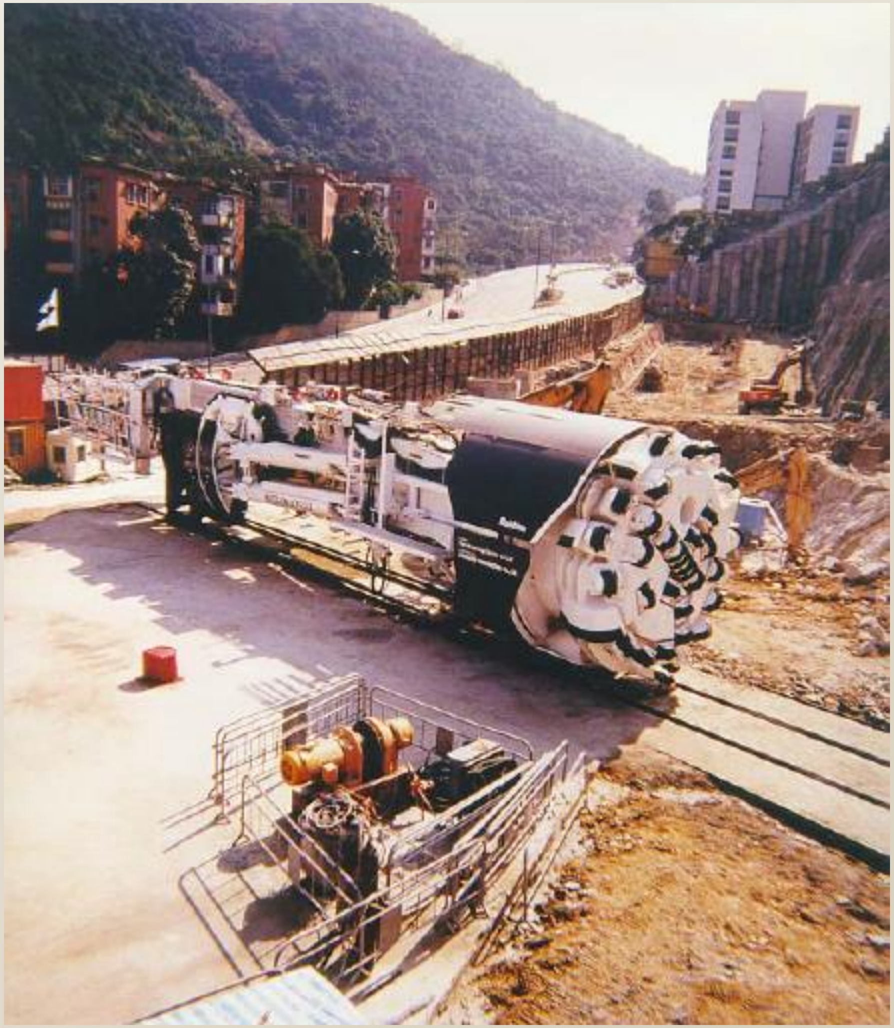
A view of the tunnel interior with the partly formed lining, tunnel supporting girders, rail track for soil disposing wagon, ventilation hose and other supply pipe lines etc.

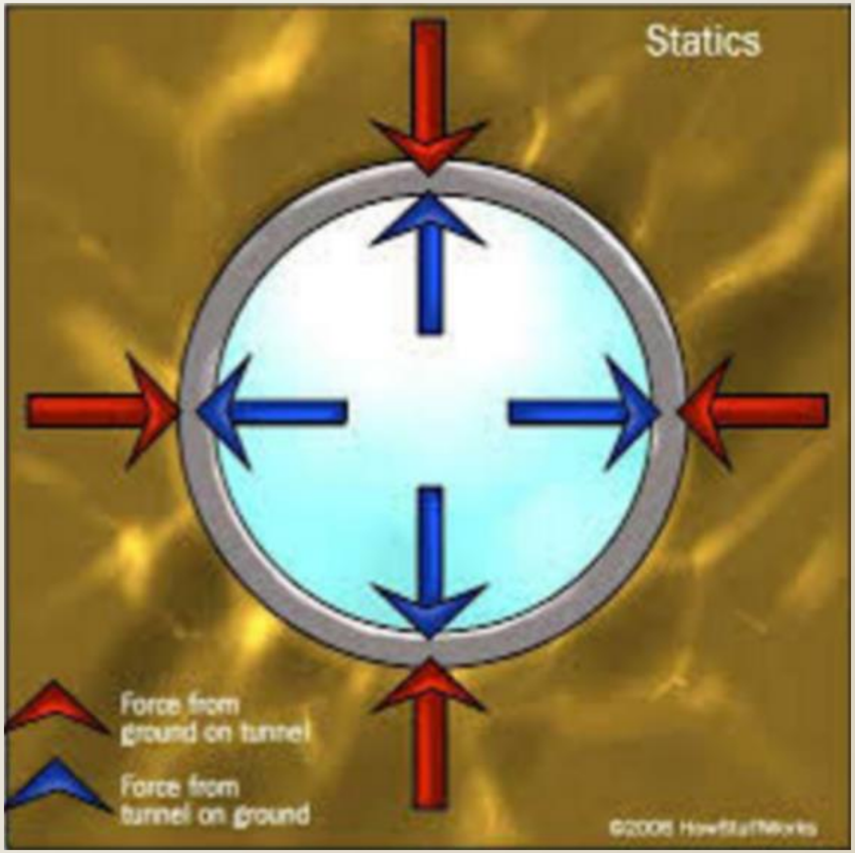
A similar tunnel boring machine employed for the forming of a cable tunnel for the Hong Kong Electric on the Hong Kong Island side. Observe the hydraulic jack systems behind the cutter head that enable the machine to stabilize itself, pushing forward, or even slight adjustment of its heading direction.



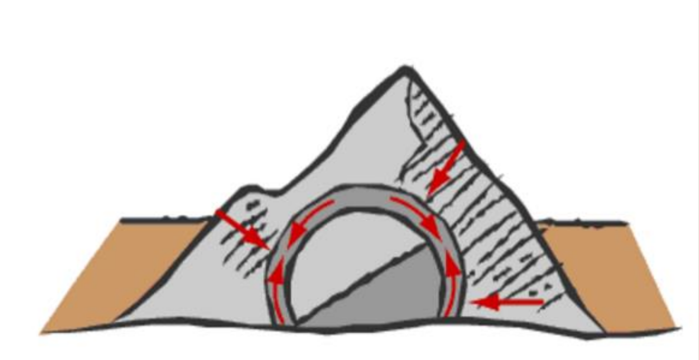
FORMATION OF THE TUNNEL PORTAL AND OTHER LOGISTIC SET-UP TO FACILITATE THE TUNNEL CONSTRUCTION WORKS

The formation of the tunnel portal for the Kwai Tsing Tunnel at Mei Foo side.

Rock Pressure



- The most important potential loads acting on underground structures are earth/rock pressures and water pressure.



- Live loads due to vehicle traffic on the surface can be safely neglected, unless the tunnel is a cut and cover type with a very small depth of overburden.

P. Randy Trabold Collection, courtesy North Adams Transcript

- It may be generally stated, that the dimensioning of tunnel sections must be effected either against the overburden weight (geostatic pressure) or against loosening pressure (i.e. the weight of the loosened zone, called also protective or Trompeter's zone).

Design Approach should Include These Elements:

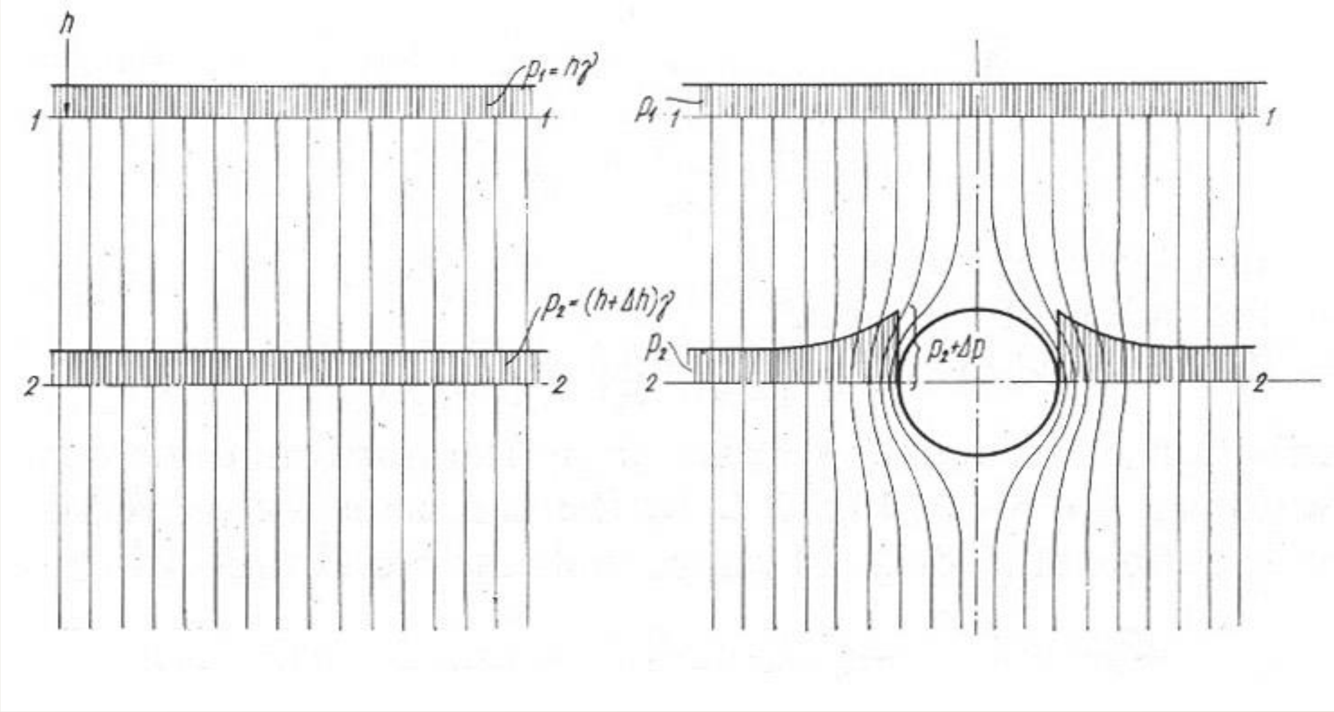
- Experience, incorporating features of empiricism based on an understanding of ground characteristics and on successful practices in familiar or similar ground.
- Reason, using analytical solutions, simple or more complex as the situation may demand, based on a comprehensive data on the ground conditions
- Observation of the behaviour of the tunnel during construction, developing into monitoring with systematic pre-designed modification of supports.

Types Of Rock Pressure

- In nature, deep lying rocks, are affected by the weight of the overlying strata and by their own weight. These factors develop stresses in the rock mass. In general every stress produces a strain and displaces individual rock particles. But to be displaced, a rock particle needs to have space available for movement. While the rock is confined, thus preventing its motion, the stresses will be accumulated or stored in the rock and may reach very high values, far in excess of their yield point.
- As soon as a rock particle, acted upon by such a stored, residual or latent stress, is permitted to move, a displacement occurs which may take the form either of 'plastic flow' or 'rock bursts' (popping) depending upon the deformation characteristics of the rock-material.
- Whenever artificial cavities are excavated in rock the weight of the overlying rock layers will act as a uniformly distributed load on the deeper strata and consequently on the roof of the cavity.

- The resisting passive forces (shear strength) are scarcely mobilized prior to the excavation of the cavity, since the deformation of the loaded rock mass is largely prevented by the adjacent rocks. By excavating the cavity, opportunity is given for deformation towards its interior.
- In order to maintain the cavity the intrusion of the rock masses must be prevented by supporting structures.
- The load acting on the supports is referred to *rock pressure*. The determination of the magnitude or rock pressure is one of the most complex problems in engineering science.
- This complexity is due not only to the inherent difficulty of predicting the *primary stress conditions* prevailing in the interior of the non-uniform rock mass, but also to the fact that, in addition to the strength properties of the rock, the magnitude of *secondary pressures* developing after excavation around the cavity is governed by a variety of factors, **such as the size of the cavity, the method of its excavation, rigidity of it support and the length of the period during which the cavity is left unsupported**.

- Rock pressures depend not only on the quality of rock and on the magnitude of stresses and strains around the cavity, but also on the amount of time elapsing after the outbreak of the underground cavity.
- Within any particular rock the pressures to which it was exposed during its history are best indicated by the pattern of folds, joints and fissures, but it is difficult to determine how far these pressures are still latent.



- According to **Terzaghi** secondary rock pressure, should be understood as the weight of a rock mass of a certain height above the tunnel, which, when left unsupported would gradually drop out of the roof, and the only consequence of installing no support props would be that this rock mass would fall into the cavity. Successive displacements would result in the gradual development of an irregular natural arch above the cavity without necessarily involving the complete collapse of the tunnel itself.

- Earth pressure, on the other hand, would denote the pressure exerted by cohesionless, or plastic masses on the tunnel supports, without any pressure relief that would, in the absence of supports, sooner or later completely fill the cavity leading to its complete disappearance.

- In general, the magnitude of earth pressure is independent of the strength and installation time of the supporting structure and it is only its distribution that is affected by the deformation of the latter. The magnitude of rock pressures, on the other hand, is influenced decisively by the strength and time of installation of props.

- This is because the deformation following the excavation of the cavity in rock masses surrounding the tunnel is of a plastic nature and extends over a period of time. This period required for the final deformations and, thus for the pressures to develop, generally increases with the plasticity of the rock and with the depth and dimensions of its cross-section. The magnitude of deformations and consequently that of stresses can, therefore, be limited by sufficiently strong propping installed at the proper time.

- It should be remembered, however, that the intensity of plastic pressures shows a tendency to decrease with increasing deformations. Furthermore the loads are carried both by the tunnel lining and the surrounding rock and every attempt should be made to utilize this cooperation.

The reasons for the development of secondary rock pressures can be classified according to **Rabcewicz** in the following three main categories:

- Loosening of the rock mass
- The weight of the overlying rock masses and tectonic forces
- Volume expansion of the rock mass, swelling due to physical or chemical action.

These reasons lead in general to the development of the following three types of rock pressure:

- Loosening pressure
- Genuine mountain pressure
- Swelling pressure

The conditions under which rock pressures develop, the probability of their occurrence and their magnitude differ greatly from one another and require the adoption of different construction methods.

Rock pressure theories

There are various rock pressure theories; one group of rock pressure theories deal, essentially, with the determination of loosening pressure since the existence of any relationship between the overburden depth and mountain pressure is neglected. The group of theories that does not take the effect of depth into account are:

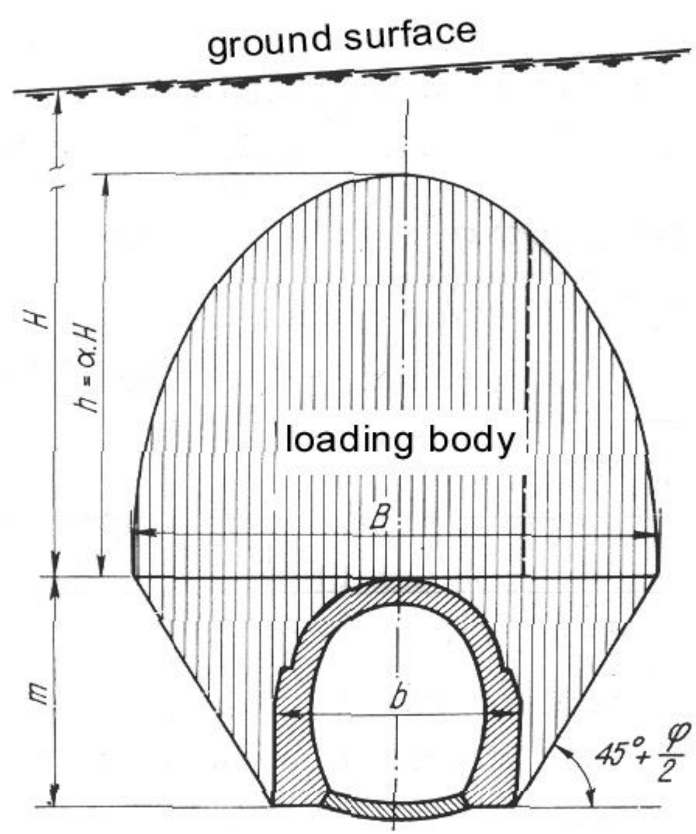
- Kommerell's theory
- Forchheimer's theory
- Ritter's theory
- Protodyakonov's theory
- Engesser's theory
- Szechy's theory

Another group of rock pressure theories takes into account the height of the overburden above the tunnel cavity. The group of theories that takes the effect of depth into account are:

- Bierbäumer's theory
- Maillart's theory
- Eszto's theory
- Terzaghi's theory
- Suquet's theory
- Balla's theory
- Jaky's Concept of theoretical slope

Vertical Loading - Bierbäumer's Theory

Bierbäumer's theory was developed during the construction of the great Alpine tunnels. The theory states that a tunnel is acted upon by the load of a rock mass bounded by a parabola of height, h = αH.



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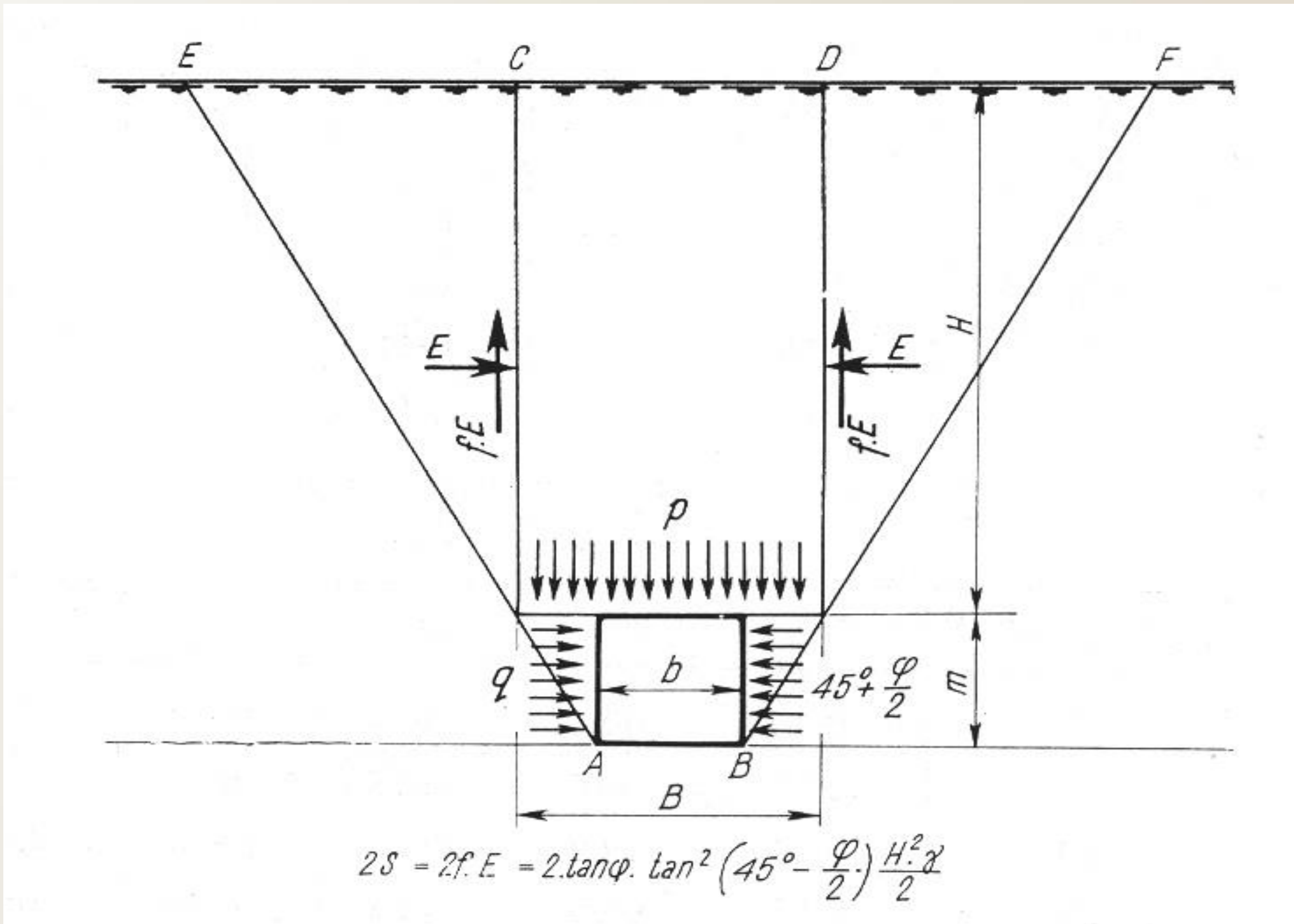
Two methods, yielding almost identical results, were developed for the determination of the value of the reduction coefficient α.

One approach was to assume that upon excavation of the tunnel the rock material tends to slide down along rupture planes inclined at $45° + \varphi/2$

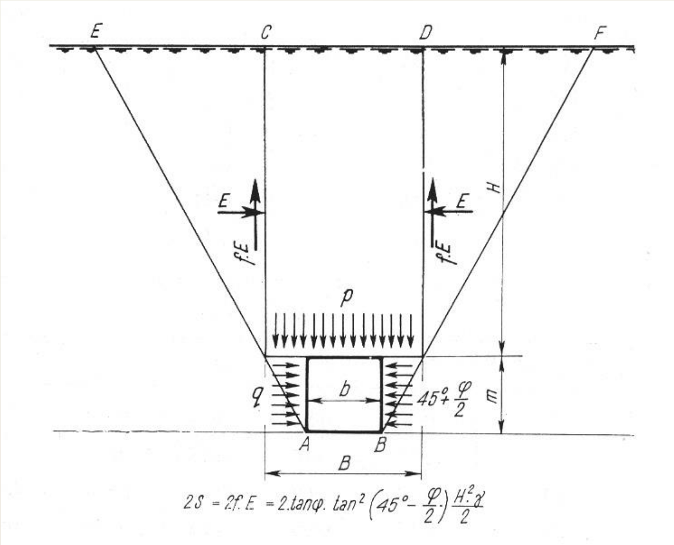


The weight of the sliding rock masses is counteracted by the friction force

$$S = 2fE = 2 \tan \Phi \tan^2 (45° - \Phi/2) \cdot$$

developing along the vertical sliding planes and therefore a rock mass of height αH only, instead of H, must be taken into account during the calculations. Consequently, the pressure on width $b + 2m \tan (45° - \Phi/2)$ at the crown will be:

$$p = \alpha 1 H \gamma$$



Taking into consideration the load diagram shown in Figure 3 the value of $\alpha 1$ is derived as follows:

$$P = H\gamma[b + 2m \cdot \tan(45° - \Phi/2)] - H^2 \gamma \tan^2 (45° - \Phi/2) \tan \Phi$$

Since

$$p = \frac{P}{b + 2m \cdot \tan(45° - \Phi/2)} = H\gamma \left[1 - \frac{\tan \Phi \cdot \tan^2 (45 - \Phi/2) H}{b + 2m \cdot \tan(45° - \Phi/2)} \right]$$

Thus

$$\alpha_1 = 1 - \frac{\tan \Phi \cdot \tan^2 (45° - \Phi/2) H}{b + 2m \cdot \tan(45° - \Phi/2)}$$

For single-track tunnels
$b = 7$ m, $m = 8$ m

$H_{(m)}$ \ ϕ	15°	20°	25	30°	35°	40°	45°
20	0·80	0·79	0·78	0·77	0·76	0·74	0·72
30	0·70	0·69	0·67	0·65	0·63	0·61	0·58
40	0·60	0·58	0·56	0·54	0·51	0·48	0·44
50	0·50	0·48	0·44	0·42	0·38	0·34	0·30
75	0·42	0·38	0·32	0·26	0·21	0·17	0·12
100	0·36	0·32	0·26	0·20	0·15	0·12	0·09
125	0·35	0·28	0·22	0·17	0·12	0·09	0·07
150	0·35	0·24	0·19	0·14	0·10	0·08	0·06
175	0·35	0·24	0·17	0·12	0·08	0·06	0·04
200	0·35	0·24	0·17	0·11	0·07	0·05	0·04
min.	0·35	0·24	0·17	0·11	0·07	0·05	0·03

For double-track tunnels
$b = 10$ m, $m = 8$ m

$H_{(m)}$ \ ϕ	15°	20°	25°	30°	35°	40°	45°
20	0·86	0·84	0·84	0·83	0·83	0·83	0·83
30	0·79	0·76	0·76	0·73	0·73	0·73	0·73
40	0·72	0·68	0·66	0·64	0·64	0·63	0·63
50	0·65	0·60	0·58	0·55	0·54	0·53	0·53
75	0·48	0·42	0·37	0·33	0·31	0·29	0·29
100	0·39	0·36	0·29	0·24	0·18	0·15	0·11
125	0·35	0·30	0·24	0·19	0·14	0·11	0·08
150	0·35	0·28	0·20	0·16	0·11	0·09	0·06
175	0·35	0·24	0·18	0·13	0·09	0·07	0·05
200	0·35	0·24	0·17	0·12	0·08	0·06	0·04
min.	0·35	0·24	0·17	0·11	0·07	0·05	0·03

implying that geostatic pressure is diminished by the friction produced by the horizontal earth pressure of the wedges EC and DF acting on the vertical shear planes.

Values of the reduction coefficient for single and double-track tunnels, as well as for various angles of internal friction Φ and depths *H* are compiled in table 1.

The reduction coefficient $\alpha 1$ has two limit values, namely for very small overburden depths $\alpha = 1$ and at several hundred metres depth, whenever $H > 5B$, the magnitude of $\alpha 1$ is no longer affected by depth and becomes

$$\alpha_1 = \tan^4 (45° - \Phi/2)$$

Terzaghi's vertical loading formula was based on a series of tests:

$$P_v = \frac{\gamma B}{2K \tan \Phi}$$

B, Φ, and γ are the same as Bierbäumer.

K= 1 (based on his tests)

Shallow Tunnel)

$$P_{roof} = \frac{\gamma B}{2K \tan\phi}(1 - e^{K \tan\phi \frac{2H}{B}})$$

where *B* is the relaxed range (=2[*b*/2+*mtan*(45−*φ*/2)]), γ is the unit weight of rock mass, *K* is coefficient of lateral earth pressure, *φ* is friction angle, *b*, *m*, and *H* are the width, height, and depth of a tunnel respectively.

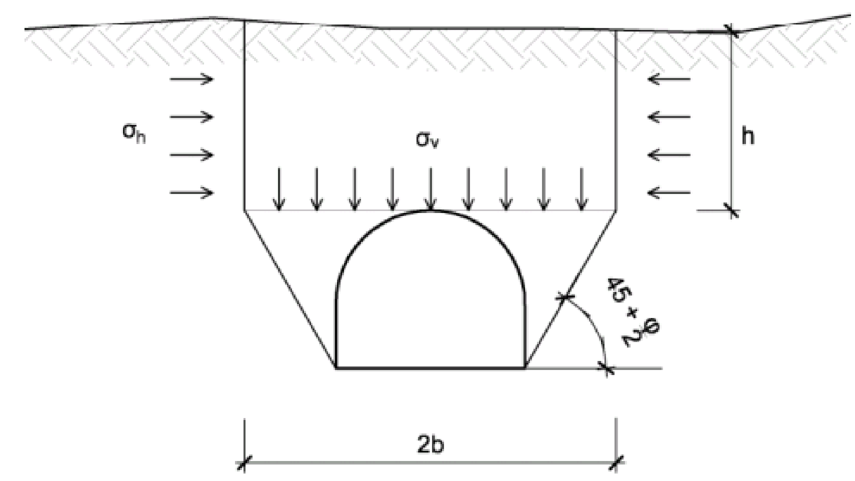


Figure 1. Terzaghi's assumption to estimate the vertical load on the tunnel support.

Deep Tunnel

$$P_{roof} = \frac{\gamma B}{2K \tan\phi}(const.)$$

Determination of Lateral Pressures on Tunnels

- The magnitude of lateral pressures has equal importance with the vertical or roof pressures for structural dimensioning of tunnel sections.
- The sidewalls of the cavity are often the first to fail owing to the less favourable structural conditions developing in the rock there. In some instances lateral pressure may play a more important role than the roof load.
- Also its theoretical estimation is more involved than that of the roof load, since its magnitude is even more affected by the extent of deformations of the section, so that its value depends increasingly on the strength of lateral support besides the properties of the rock and dimensions of the cavity.
- Lateral pressures are also much more affected by latent residual geological stresses introduced into the rock mass during its geological history which are released upon excavation and whose magnitude depends on the deformation suffered by, and the elasticity of the rock, but it is unpredictable. Genuine mountain pressure and swelling pressure that cannot be evaluated numerically may act in full on the sidewalls.

Approximate determination of lateral pressures:

- Lateral pressures in soils are determined approximately from earth pressure theory, as a product of geostatic pressure, or roof load and the earth pressure coefficient, respectively in terms of the lateral strain.
- Lateral pressures are (Stini) in contrast to roof loads, in a linear relationship with overburden depth. This has been confirmed also by recent model tests and in-situ stress measurements.
- At greater depths pressures on the springings are usually higher than on the roof, but at the same time frictional resistance is also higher.

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Lateral pressures compiled according to practical experience are given in the table below:

OBSERVED ROCK-PRESSURE VALUES

Rock material	Roof pressure p_v (t/m²)		Lateral pressure $p_h = \frac{1}{2} p_v - \frac{1}{3} p_v$		Bottom pressure (t/m²)	Temporary timber support		Remark
	At outbreak	After completion of drift	Initial (t/m³)	Outbreak completed (t/m²)		Mode of execution	Degree of stressing	
Rock, more or less blocky	0	8–12	—	—	—	Skeleton lagging, light	0 to insignificant	Loosening pressure small
Very seamy rock, cemented conglomerate, soft rock, with small overburden height	10	30	—	3	4–6	Skeleton lagging, solid	Small	Loosening pressure increasing at the moment of outbreak not perceivable
Heavily fractured rock (roof breakdown) rolling gravel and conglomerate	15–25	30–40	5–10	5–15	10	Tight, strong lagging	Mean	Bigger pressures perceivable simultaneously with outbreak. Ensuing of equilibrium condition, very prolongated
Loose rock under heavy pressure (eventually in saturated condition). Bigger overburden height	25–35	40–60	10	10	15	Very tight solid lagging	Considerable	Stabilization of pressure conditions very difficult
Loose and soft (pseudosolid) rock under heavy pressure. Very big overburden height.	40–60	100–150	20	15	30	Very tight lagging and strong hard-wood sill-beams	Going up to rupture	Stabilization possible only after the completion of very protracted deformations (months even years; Karawanken tunnel)

- According to Terzaghi, a rough estimate of lateral pressure is given by the following formula:

$$P_h = 0.3\gamma(0.5m + h_p)$$

- where *hp* is the height of the loosening core representing the roof load; in granular soils and rock debris, on the basis of Rankine's ratio

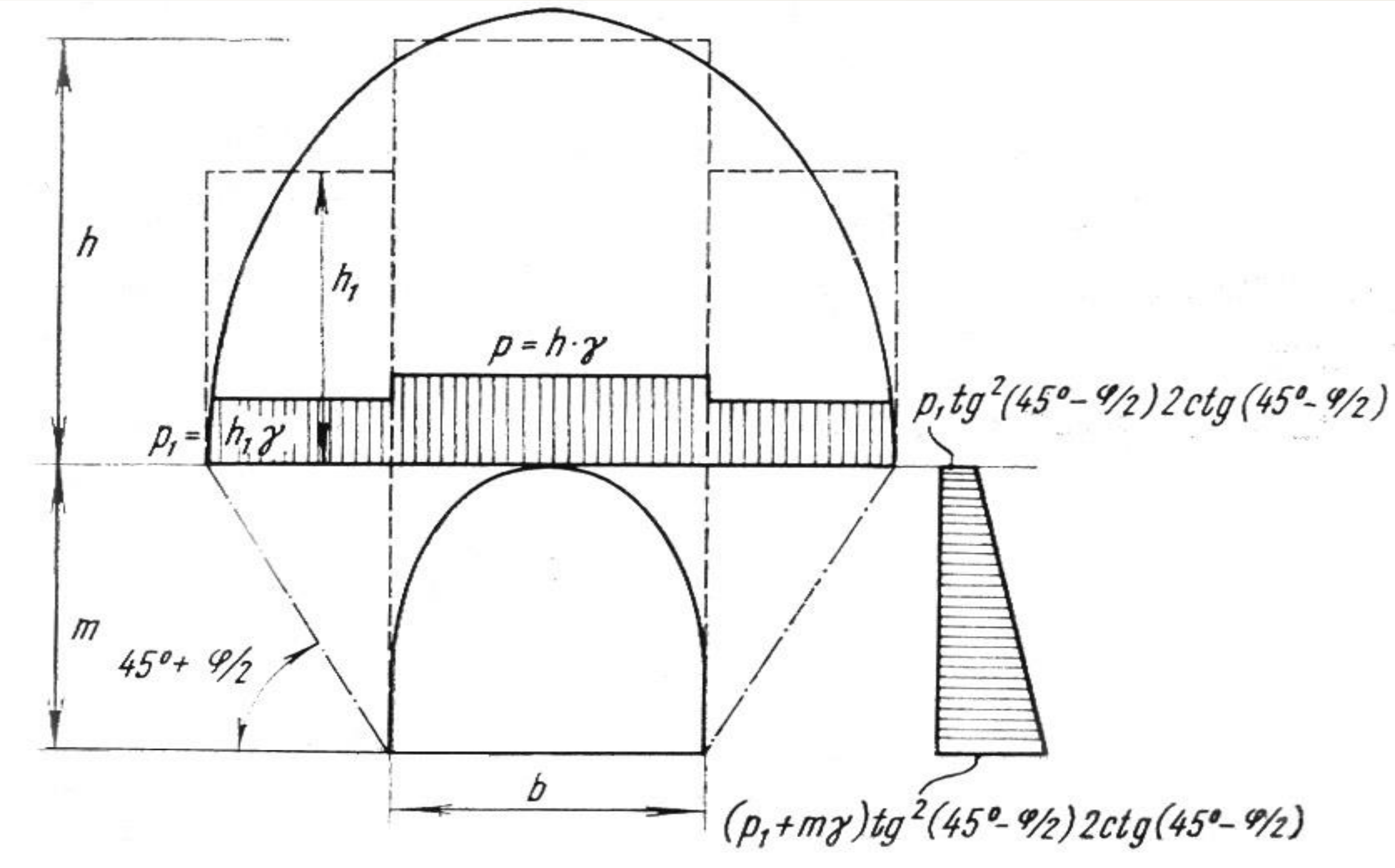
$$P_h = \gamma H \tan^2(45° - \Phi/2)$$

- and finally, in solid rocks, relying on Poisson's ratio

$$P_h = \frac{\mu}{1-\mu} p_v$$

- Lateral pressures should be assumed in a linear distribution and should be based on the vertical pressure estimated by one of the rock pressure theories instead of on geostatic pressure.

The parabolic distribution shown in below figure should be assumed for the vertical pressure having a peak ordinate corresponding to the estimated roof load.



If the pressure ordinate of the parabola an the vertical erected at the side of the cavity is , then the lateral pressure intensity at roof level will be

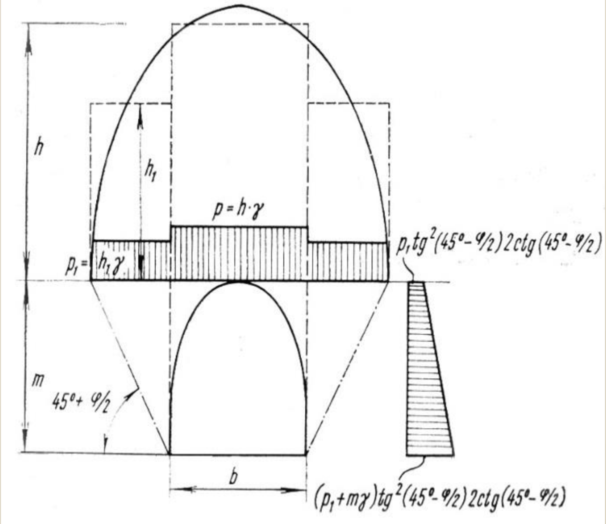
$$e_1 = p_2 \tan^2(45° - \Phi/2) - 2c\tan(45° - \Phi/2)$$

and at invert level

$$e_2 = (p_2 + m\gamma)\tan^2(45° - \Phi/2) - 2c\tan(45° - \Phi/2)$$

Owing to the favourable effect of lateral pressure on bending moments arising in the section, cohesion, which tends to reduce the magnitude of this pressure, must not be neglected in the interest of safety.

The lateral pressure coefficients involved in the above formulae are either



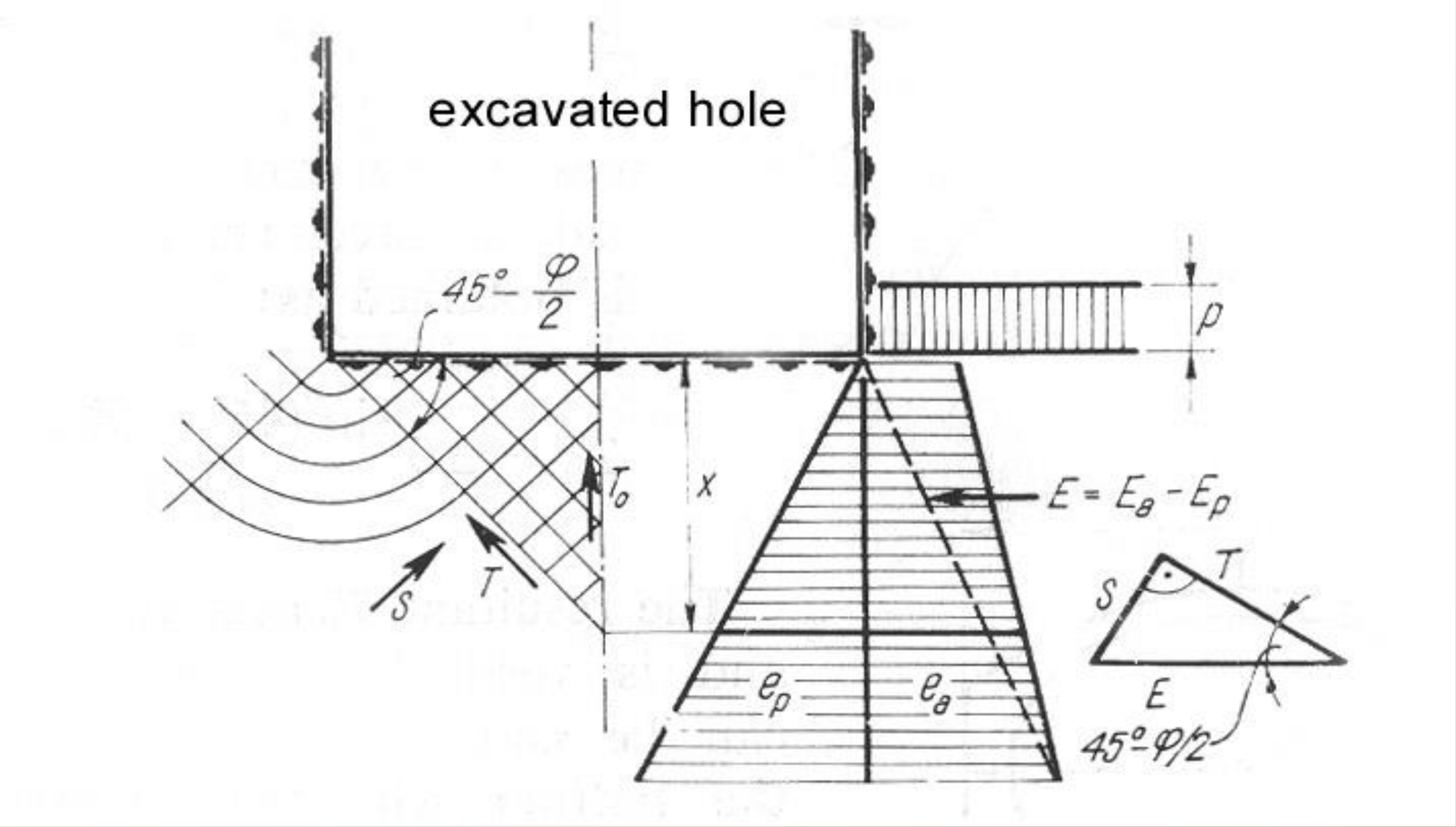
$$\lambda = \frac{\mu}{1-\mu} \quad \text{or} \quad \lambda = \tan^2(45° - \Phi/2)$$

Bottom Pressures

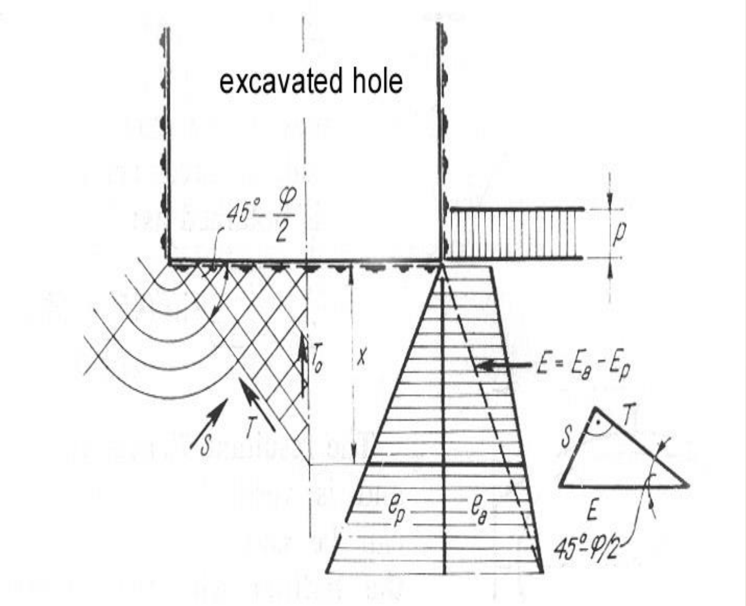
- Bottom pressures should be essentially the counterparts of roof loads, i.e. reactions acting on the tunnel section from below if the tunnel section is a closed one having an invert arch.
- A certain part of this load is, however, carried by the surrounding rock masses, so that this situation does not occur even in the case of closed sections, and bottom pressures have usually been found to be smaller than roof loads.
- Terzaghi quoted empirical evidence indicating that bottom pressures are approximately one half and lateral thrusts one-third of the roof load intensity.
- In the case of sections open at the bottom, i.e. having no invert arch, pressures of different intensity develop under the side walls and under the unsupported bottom surface. *The pressures arising under the solid side walls must be compared with the load-bearing capacity, or ultimate strength of the soil, but do not otherwise affect the design of the tunnel, The magnitude of rock pressure acting upward towards the interior of the open tunnel section is, however, unquestionably affected by these pressures .*
- The development, distribution and magnitude of bottom pressures are greatly influenced by the method of construction adopted, i.e. by the sequence in which various structures and components of the tunnel are completed.

Bottom pressure according to Tsimbaryevitch

He assumed that a soil wedge is displaced towards the cavity under the action of active earth pressure originating from the vertical pressure on the lateral parts. This displacement is resisted by the passive earth pressure on the soil mass lying under the bottom of the cavity.



- The active earth-pressure diagram at the perpendicular of the corner point of the excavated cavity is a trapeze. The earth pressure at depth *x* will be



$$e_a = (p + x\gamma)\tan^2(45° - \Phi/2) - 2c\tan(45° - \Phi/2)$$

- At the same time the specific passive earth pressure at depth x is

$$e_p = x\gamma\tan^2(45° - \Phi/2) + 2c\tan(45° - \Phi/2)$$

- Depth x: where ea = ep can be computed by equating the above two expressions. The layers above this depth will be involved in bottom pressure:

$$x = \frac{p\tan^2(45° - \Phi/2) - 2c\left[\tan(45° + \Phi/2) + \tan(45° - \Phi/2)\right]}{\gamma\left[\tan^2(45° + \Phi/2) - \tan^2(45° - \Phi/2)\right]}$$

and

$$x = \frac{p\lambda_a - 2c(\sqrt{\lambda_p} + \sqrt{\lambda_a})}{\gamma(\lambda_p - \lambda_a)}$$

The magnitude of the horizontal force acting towards the cavity above depth *x* is given as the difference between the areas of the diagrams for e_a and e_p. This force induces a set of sliding surfaces inclined at (45° - Φ/2) to develop in the soil mass under the cavity.

The force of magnitude E = Ea - Ep may be resolved into components *T* and *S*, parallel to the sliding surfaces and perpendicular to them, respectively:

T = E cos (45º - Φ/2)

S = E sin (45º - Φ/2)

Force *T* tends to displace the soil and is resisted by the frictional component of the normal force

T = *S* tan Φ

After trigonometric transformations and remembering that the soil is displaced by forces acting from both corners, the magnitude of forces acting on the bottom plane is obtained as:

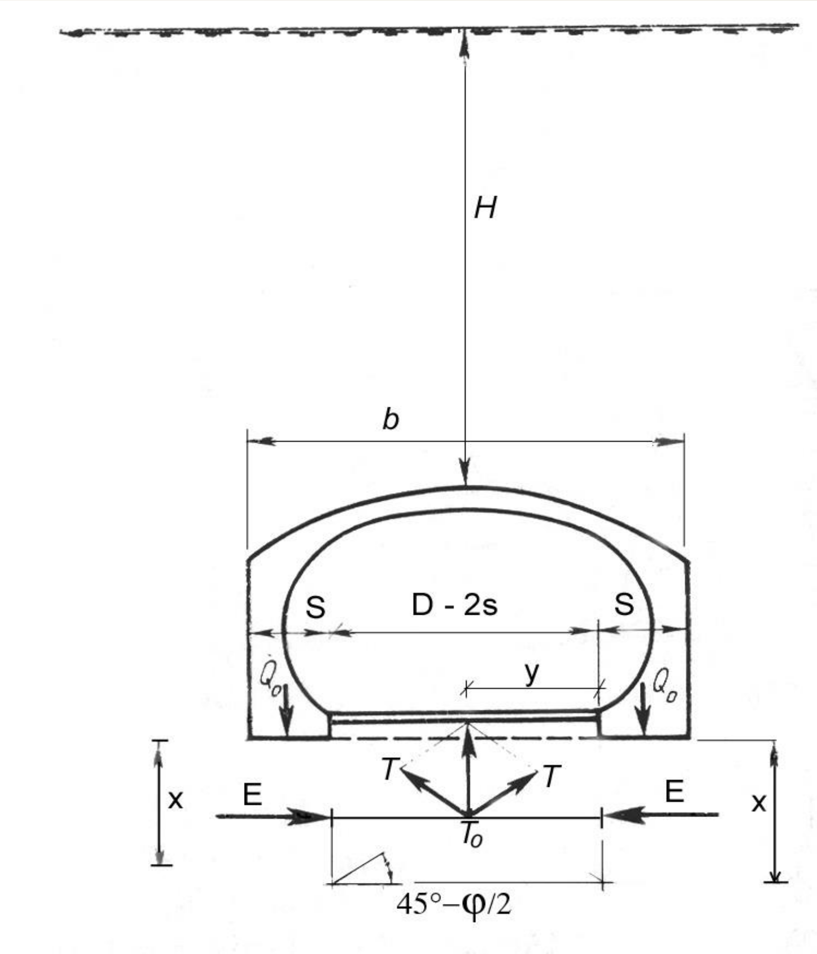
$$T_0 = 2E\frac{\sin^2(45° - \Phi/2)}{\cos\Phi}$$

The resultant T_o acts at the centre line and is vertical. This upward pressure can be counteracted either by loading the bottom with the counterweight of intensity qo, or by a suitably dimensioned invert arch.

The counter load qo must be applied over a length y, which can be obtained from the expression:

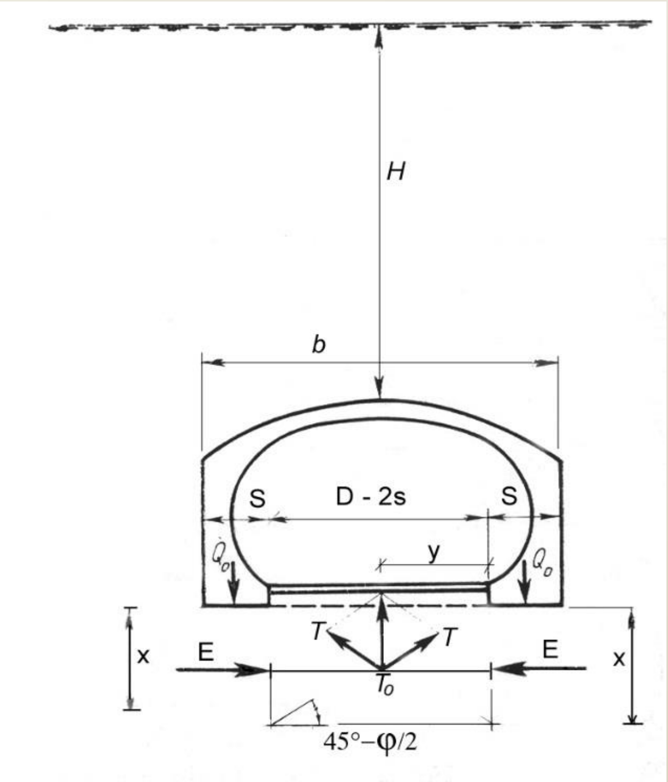
$$y = \frac{x}{\tan(45^\circ - \Phi / 2)}$$

The pressure acting on the bottom of the cavity in the practical case illustrated in the figure below.



Assuming a granular soil, it can be determined in the following way:

If the bottom re action under the side walls is $p = \frac{Q_0}{s}$, the height of the soil column at the side of the cavity can be obtained from the relation $H = \frac{q}{y}$



Since

$$x = H \frac{\tan^2(45° - \Phi/2)}{\tan^2(45° + \Phi/2) - \tan^2(45 - \Phi/2)}$$

And

$$E = E_a - E_p = \frac{1}{2}\gamma x(x + 2H)\tan^2(45° - \Phi/2) - \frac{1}{2}\gamma x^2 \tan^2(45° + \Phi/2)$$

The pressure acting from below on the cavity is

$$T = E\frac{\sin(45° - \Phi/2)}{\cos\Phi}$$

$$T_0 = 2T\sin(45° - \Phi/2)$$

Closure of the section with a bottom slab and the application of an internal ballast are the only possible counter-measures to this pressure. In the interest of safety a coefficient n = 1.3 to 1.5 has been specified.

With $\frac{b}{2} - s$ denoting the actual loading width, p s and P_b the weight of the bottom slab and internal ballast, respectively, the coefficient of safety can be determined by comparing the resulting downwards stress and the upward pressure *N*/*y*. In other words, it is required that

$$n = \frac{p_s + P_b}{T_0} \frac{y}{\frac{b}{2} - 2} \geq 1{,}3 - 1{,}5$$

LESSON 5.
STRESSES AROUND UNDERGROUND EXCAVATIONS

Introduction

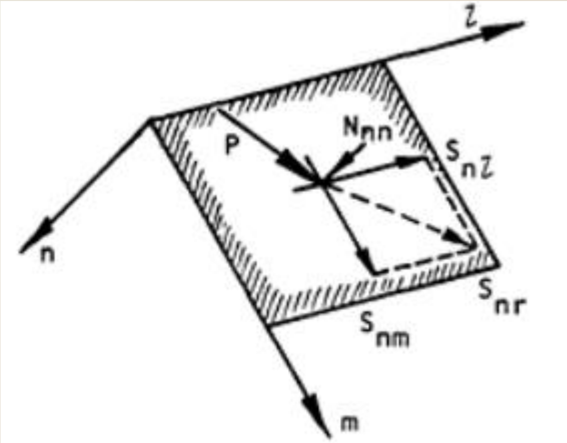
- The stresses which exist in an undisturbed rock mass are related to the weight of the overlying strata and the geological history of the rock mass.
- This stress field is disturbed by the creation of an underground excavation and, in some cases, this disturbance induces stresses which are high enough to exceed the strength of the rock.
- In these cases, failure of the rock adjacent to the excavation boundary can lead to instability which may take the form of gradual closure of the excavation, roof falls and slabbing of sidewalls or, in extreme cases, rockbursts.
- Rockbursts are explosive rock failures which can occur when strong brittle rock is subjected to high stress.

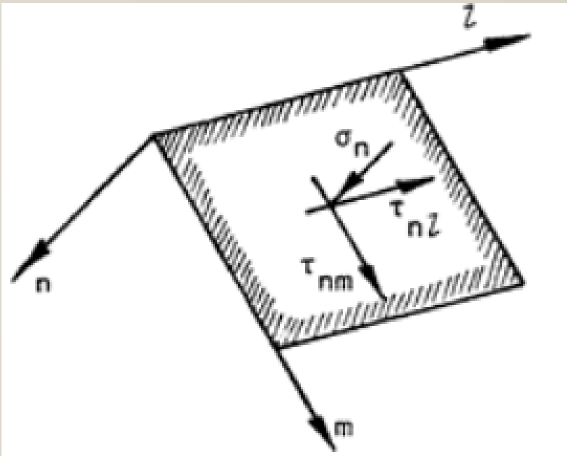
Components of stress

- Stresses are defined in terms of the forces acting at a point or on a surface. Consider the forces acting on an inclined surface within a rock mass. This surface may be
  1) an external surface forming part of the boundary of a structure,
  2) an internal structural feature such as a joint or fault, or
  3) an imaginary internal surface.

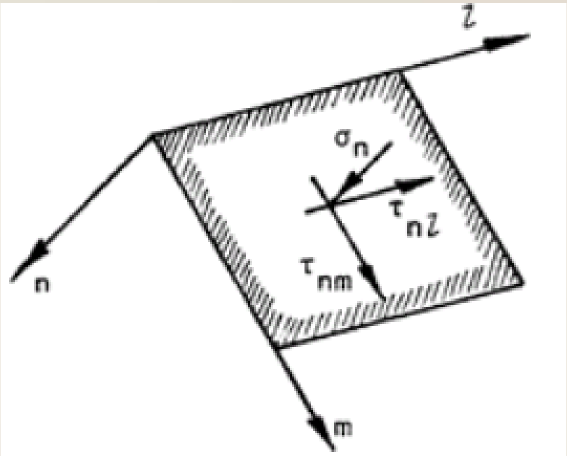
Components of stress

In general, the distribution of forces over this surface will vary, and it is therefore convenient to consider the forces applied to a small rectangular element of the surface as shown in the margin sketch. Axes *l* and m are set up parallel to the sides of the element, and axis n in the direction of the normal to the surface.





Stress at a point

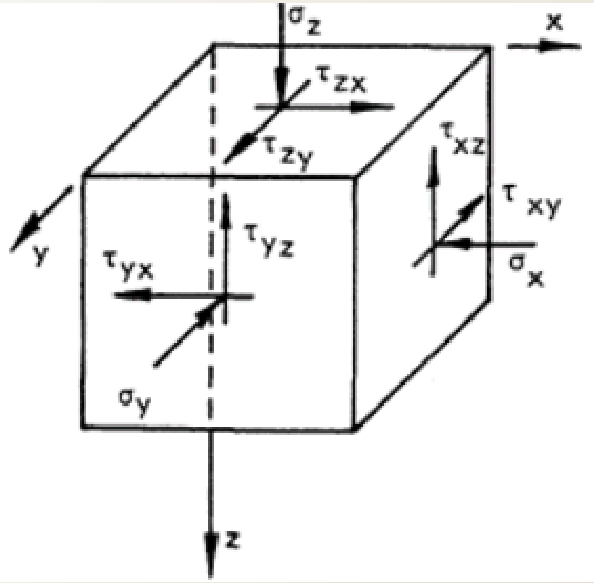


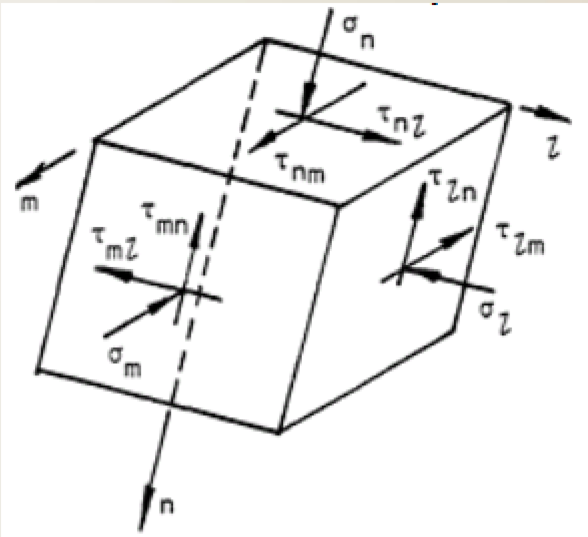
The intensity of a force applied to a surface element is obtained by dividing the force by the area of the element, A.

In terms of the component forces defined above, we write

$$\sigma_n = \frac{N_n}{A} \qquad \tau_{nl} = \frac{S_{nl}}{A} \qquad \tau_{nm} = \frac{S_{nm}}{A}$$

Stress at a point

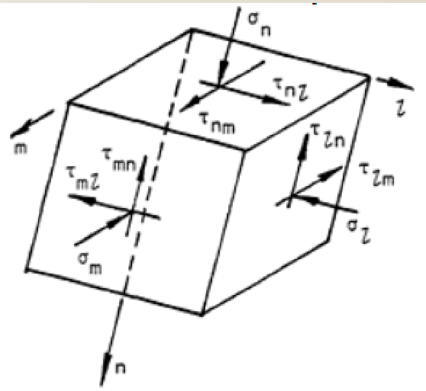




- The margin sketch shows such an element chosen with its edges parallel to the x, y and z axes. The surface tractions shown on the three visible faces are all positive. For the vertical face parallel to the y–z plane, $+\sigma_x$ acts in the negative x direction and $+\tau_{xy}$ and $+\tau_{xz}$ act in the negative y and z directions repectively, because the inwards normal to this face acts in the negative x direction. x direction and $+\tau_{xy}$ and $+\tau_{xz}$ act in the negative y and z directions repectively, because the inwards normal to this face acts in the negative x direction.

Transformation equations

It often happens that the engineer knows the components of stress referred to one set of axes (x, y, z) and wishes to determine another set of components (*l*, m, n).



$$\begin{bmatrix} \sigma_l & \tau_{lm} & \tau_{nl} \\ \tau_{lm} & \sigma_m & \tau_{mn} \\ \tau_{nl} & \tau_{mn} & \sigma_n \end{bmatrix} = \begin{bmatrix} l_x & l_y & l_z \\ m_x & m_y & m_z \\ n_x & n_y & n_z \end{bmatrix} \begin{bmatrix} \sigma_x & \tau_{xy} & \tau_{zx} \\ \tau_{xy} & \sigma_y & \tau_{yz} \\ \tau_{zx} & \tau_{yz} & \sigma_z \end{bmatrix} \begin{bmatrix} l_x & m_x & n_x \\ l_y & m_y & n_y \\ l_z & m_z & n_z \end{bmatrix}^T$$

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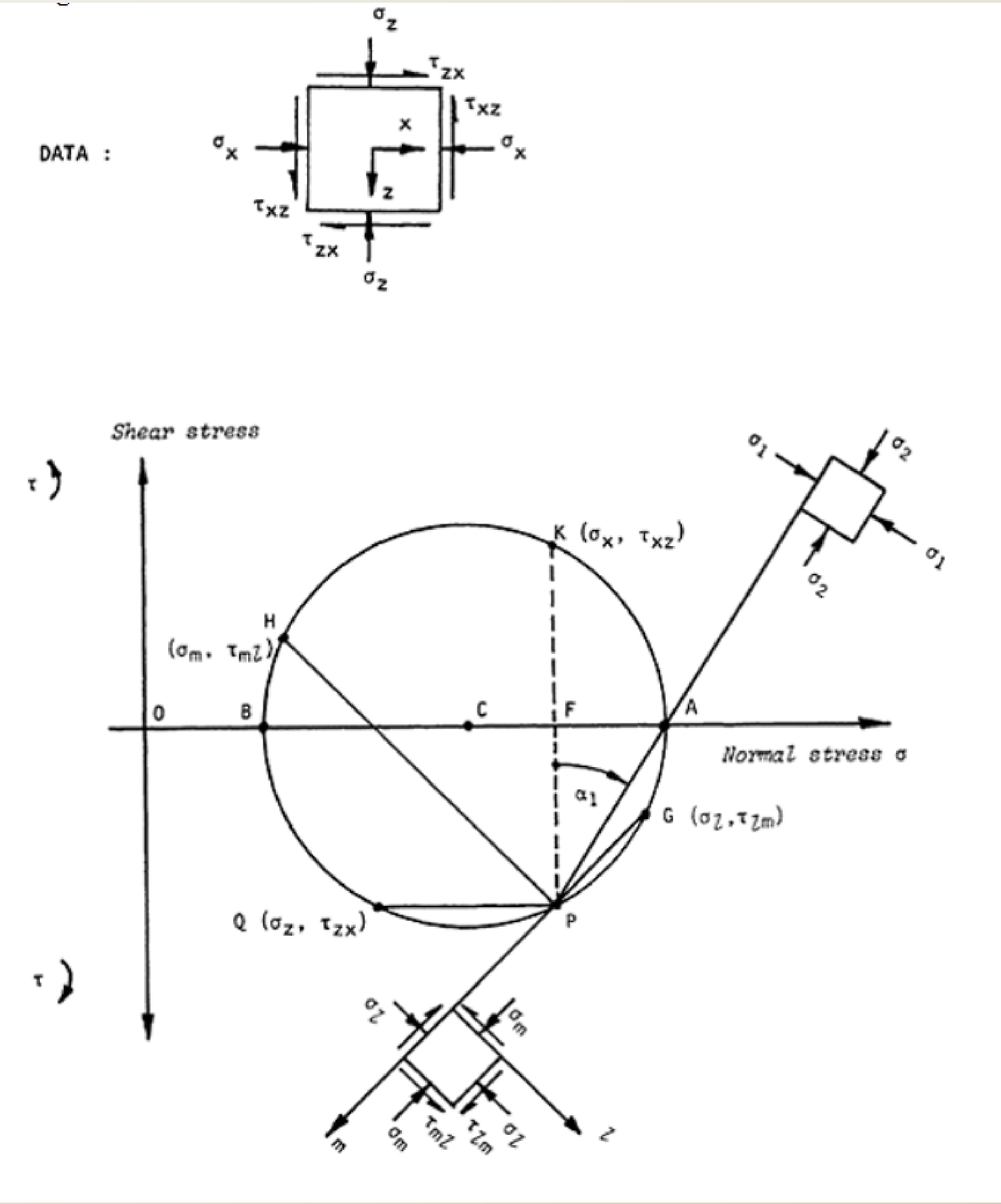
Mohr's circle diagram



Vertical stress σ_z and lateral compressive stresses

Terzaghi and Richart (1952) suggested that in the case of sedimentary rocks in geologically undisturbed regions where the strata were built up in horizontal layers in such a way that the horizontal dimensions remained unchanged, the lateral stresses σ_x and σ_y are equal and are given by putting $\varepsilon_x = \varepsilon_y = 0$.

$$\sigma_x = \sigma_y = \frac{\nu}{1-\nu}\sigma_z$$

For a typical rock having a Poisson's ratio v=0.25, equation on above shows that the lateral stresses σ_x and σ_y are each equal to one third of the vertical stress σ_z, provided that there has been no lateral strain

Vertical stress σ_z and lateral compressive stresses

- The measured vertical stresses are in fair agreement with the simple prediction given by calculating the vertical stress due to the overlying weight of rock at a particular depth from the equation

$$\sigma_z = \gamma z$$

- where γ is the unit weight of the rock (usually in the range 20 to 30 kN/m^3) and z is the depth at which the str.ess is required

In situ horizontal stress

- The horizontal stresses acting on an element of rock at a depth z below the surface are much more difficult to estimate than the vertical stresses.
- Normally, the ratio of the average horizontal stress to the vertical stress is denoted by the letter **k** such that:

$$\sigma_h = k\,\sigma_v = k\,\gamma\,z$$

In situ horizontal stress

- Terzaghi and Richart (1952) suggested that, for a gravitationally loaded rock mass in which no lateral strain was permitted during formation of the overlying strata, the value of **k** is independent of depth and is given by:

$$k = \nu/(1 - \nu)$$

  ν = The Poisson's ratio of the rock mass
- This relationship was widely used in the early days of rock mechanics but it proved to be inaccurate and is seldom used today.

In situ horizontal stress

- Measurements of horizontal stresses at civil and mining sites around the world show that the ratio **k** tends to be high at shallow depth and that it decreases at depth (Brown & Hoek, 1978; Herget, 1988).
- In order to understand the reason for these horizontal stress variations it is necessary to consider the problem on a much larger scale than that of a single site.

In situ horizontal stress

- Sheorey (1994) developed an elasto-static thermal stress model of the earth.
- This model considers curvature of the crust and variation of elastic constants, density and thermal expansion coefficients through the crust and mantle.
- He did provide a simplified equation which can be used for estimating the horizontal to vertical stress ratio **k**.

In situ horizontal stress

- This equation is:

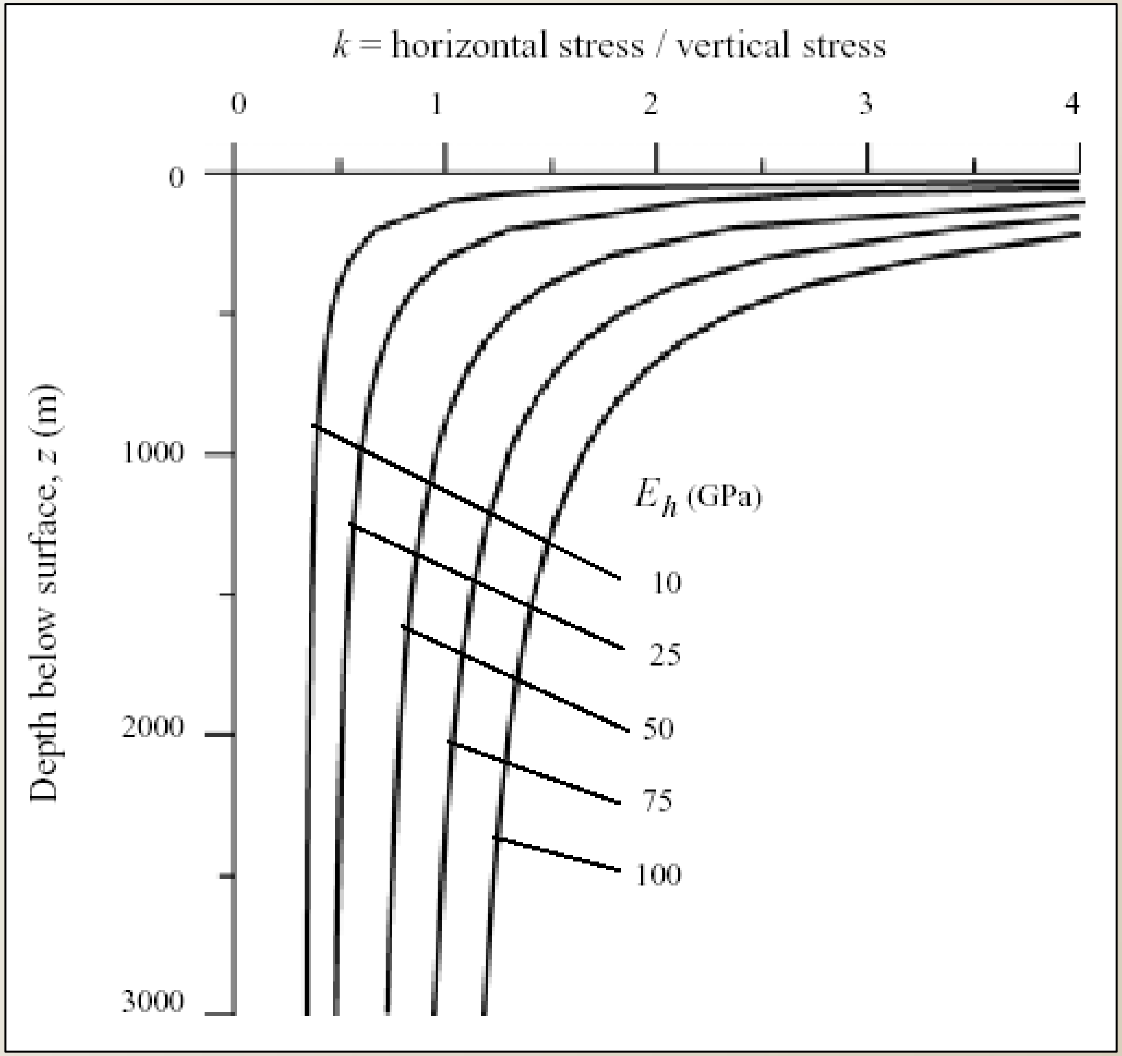
$$k = 0.25 + 7E_h\left(0.001 + \frac{1}{z}\right)$$

z (m) = The depth below surface

E_h (GPa) = The average deformation modulus of the upper part of earth's crust measured in horizontal direction

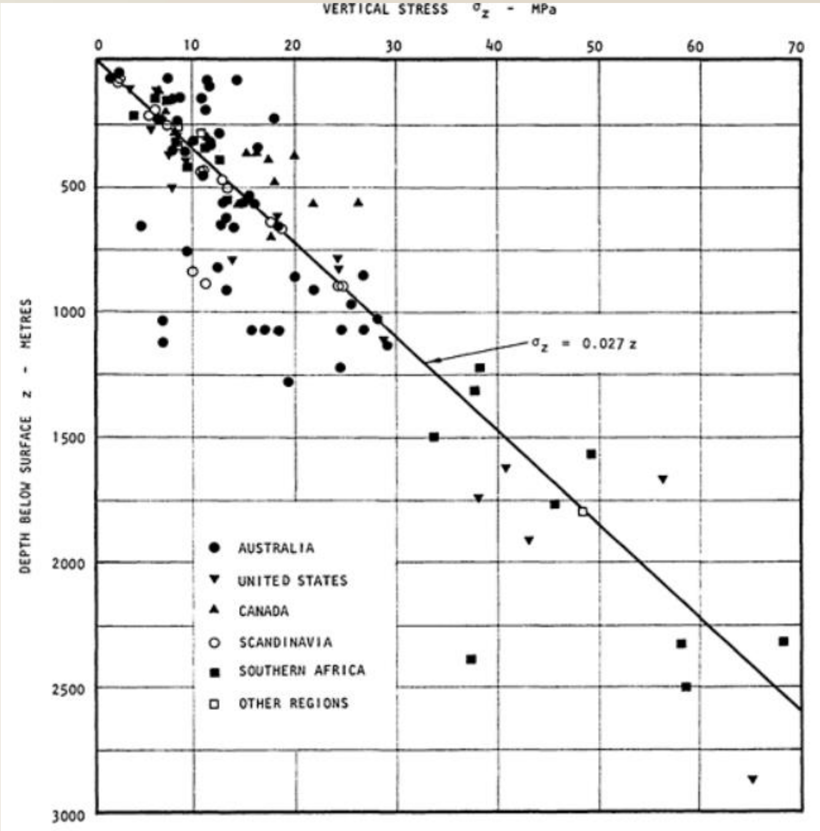
→ special caution in layered sedimentary rocks (**Why?**)

In situ horizontal stress



Vertical stress σ_z and lateral compressive stresses

At shallow depths, there is a considerable amount of scatter which may be associated with the fact that these stress values are often close to the limit of the measuring accuracy of most stress measuring tools. On the other hand, the possibility that high vertical stresses may exist cannot be discounted, particularly where some unusual geological or topographic feature may have influenced the entire stress field.



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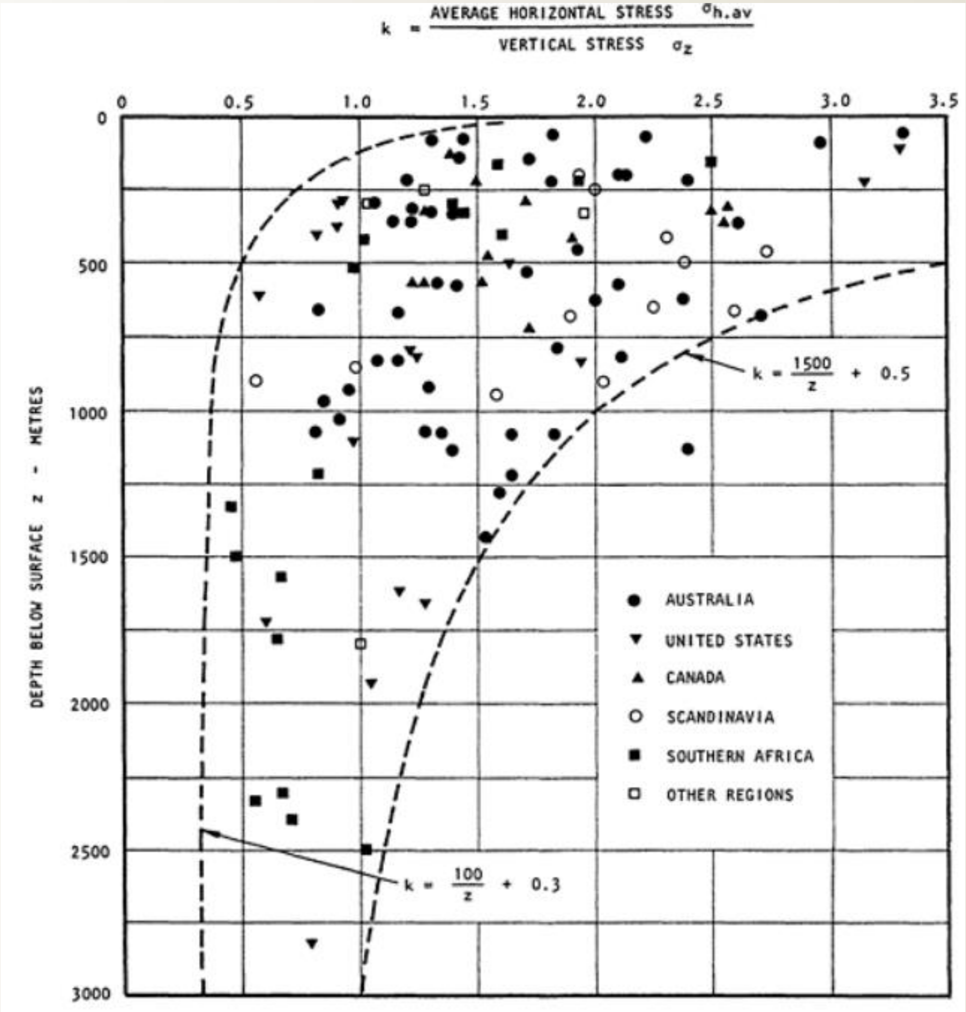
Variation of ratio of average horizontal stress to vertical stress with depth below surface



$$\frac{100}{z} + 0.3 < k < \frac{1500}{z} + 0.5$$

Stress distributions around single excavations

- ***The streamline streamline analogy for principal stress trajectories***
- When an underground excavation is made in a rock mass, the stresses which previously existed in the rock are disturbed, and new stresses are induced in the rock in the immediate vicinity of the opening.
- One method of representing this new stress field is by means of *principal stress trajectories* which are imaginary lines in a stressed elastic body along which principal stresses act.
- Before considering in detail the distribution of stresses around single underground excavations of various cross-sections, it may be helpful to the reader to visualise the stress field by making use of the approximate analogy which exists between principal stress trajectories and the streamlines in a smoothly flowing stream of water.

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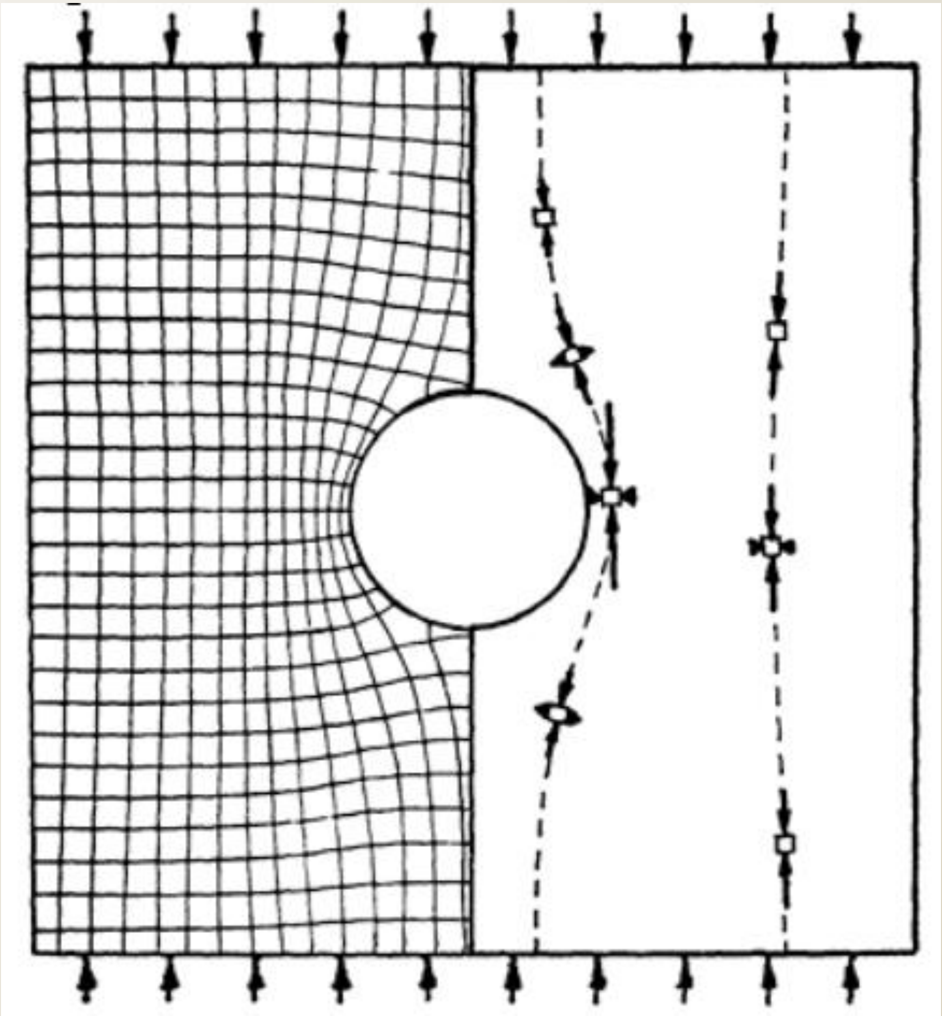
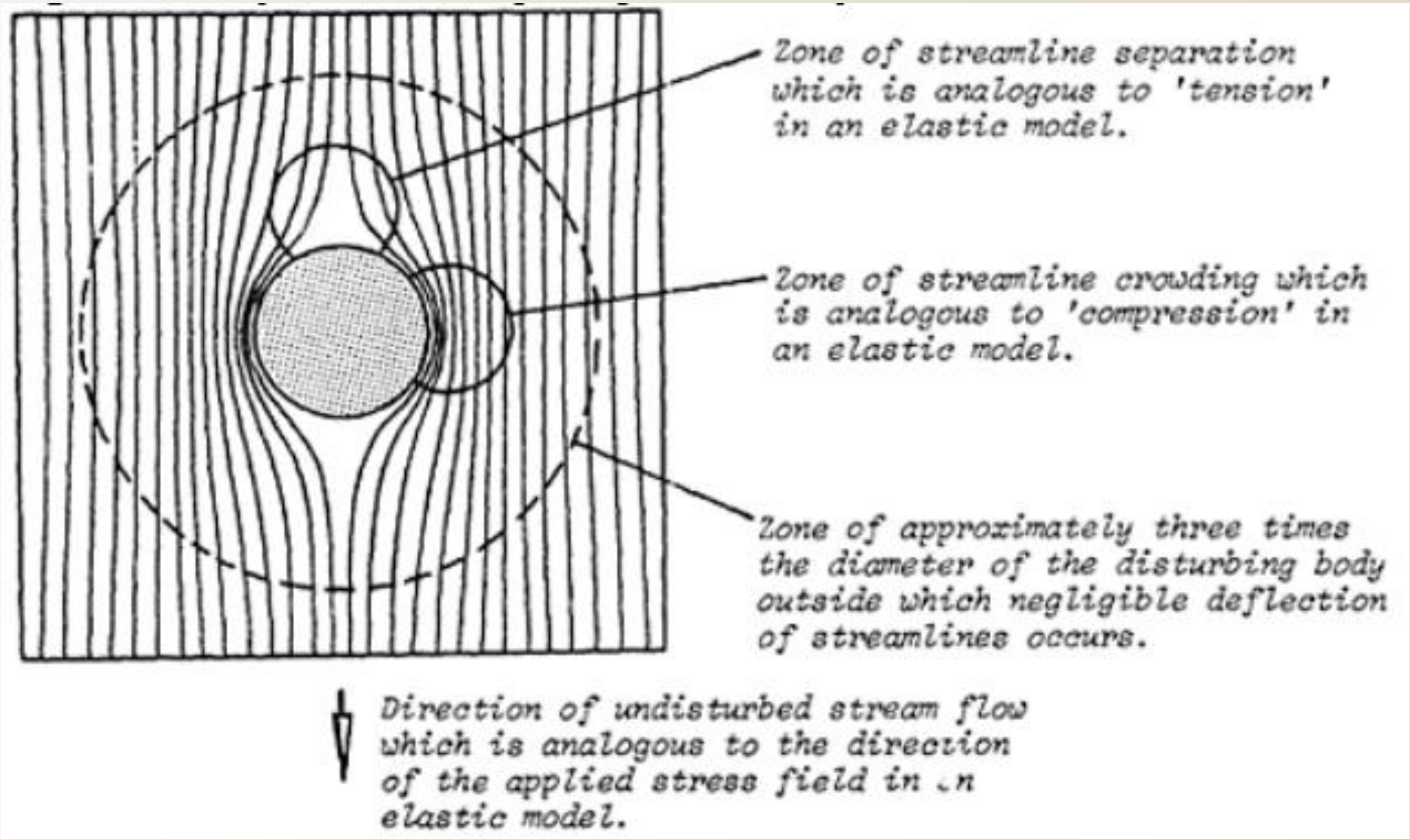
Major and minor principal stress trajectories in the material surrounding a circular hole in a uniaxially stressed elastic plate.

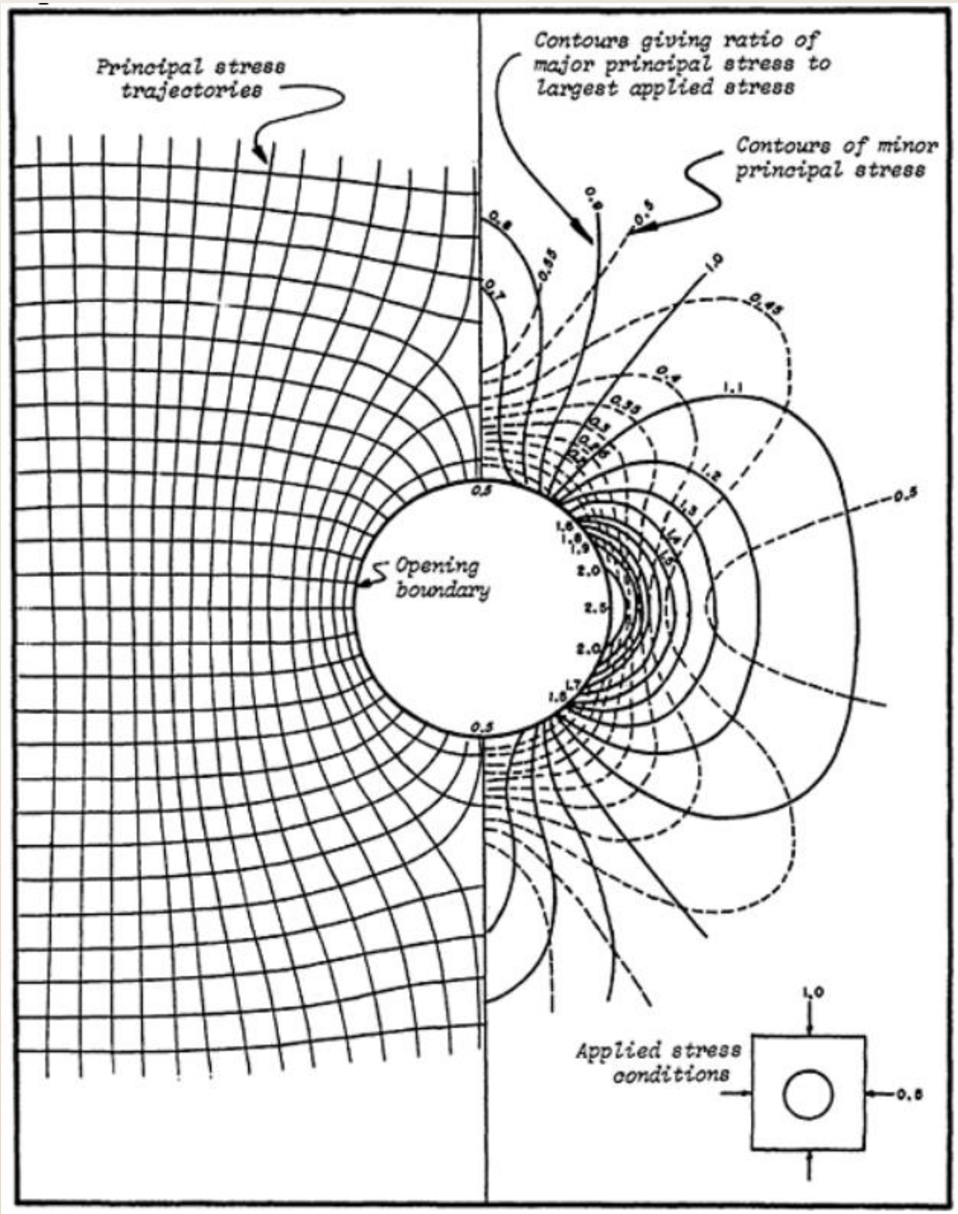


Deflection of streamlines around a cylindrical obstruction



Principal stress contours and principal stress trajectories in the material surrounding a circular hole in a stressed elastic body. As shown in the inset diagram, the ratio of applied stresses k=0.5. Solid lines are major principal stress contours and dashed lines are minor principal stress contours. Contour values are ratios of principal stresses to the larger of the two applied stressesPrincipal stress contours and principal stress trajectories in the material surrounding a circular hole in a stressed elastic body.

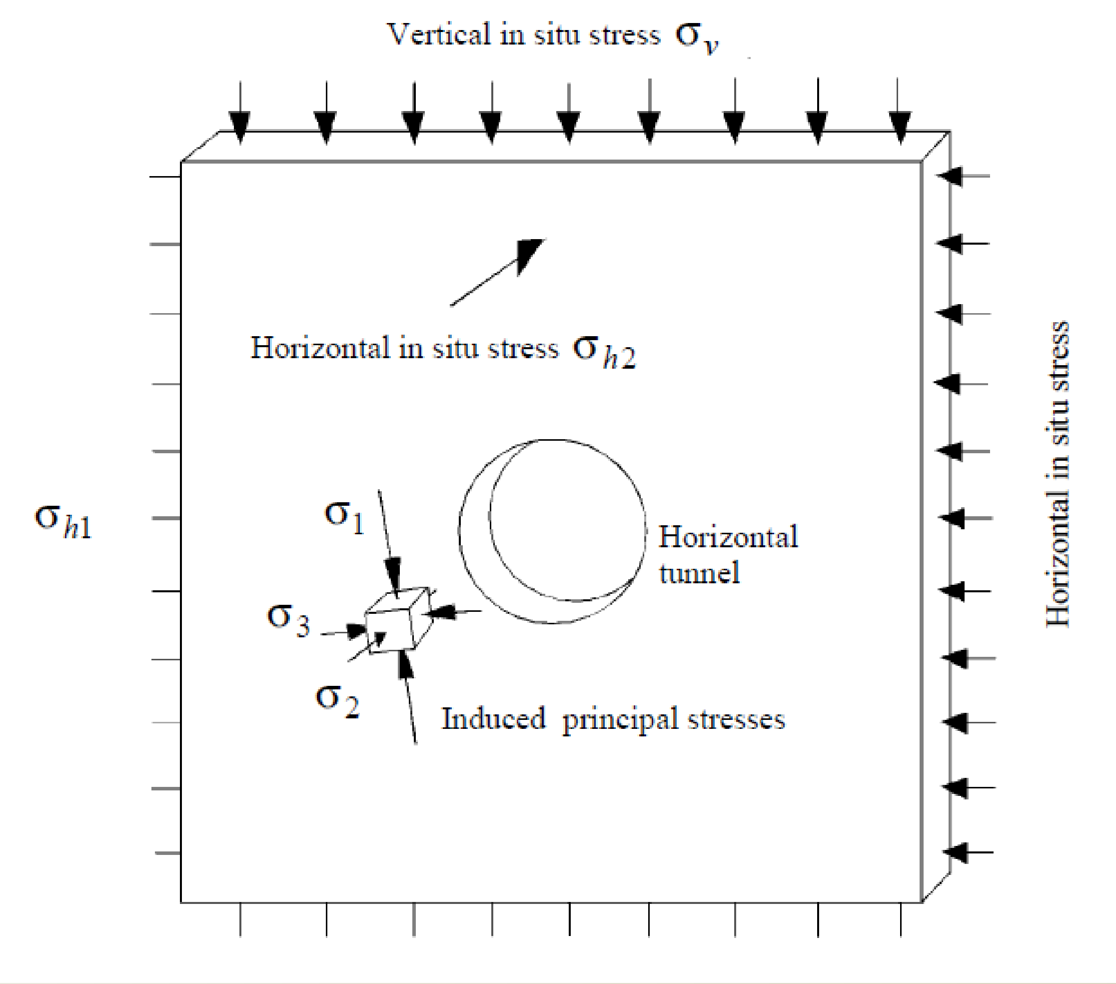
As shown in the inset diagram, the ratio of applied stresses k=0.5. Solid lines are major principal stress contours and dashed lines are minor principal stress contours. Contour values are ratios of principal stresses to the larger of the two applied stresses



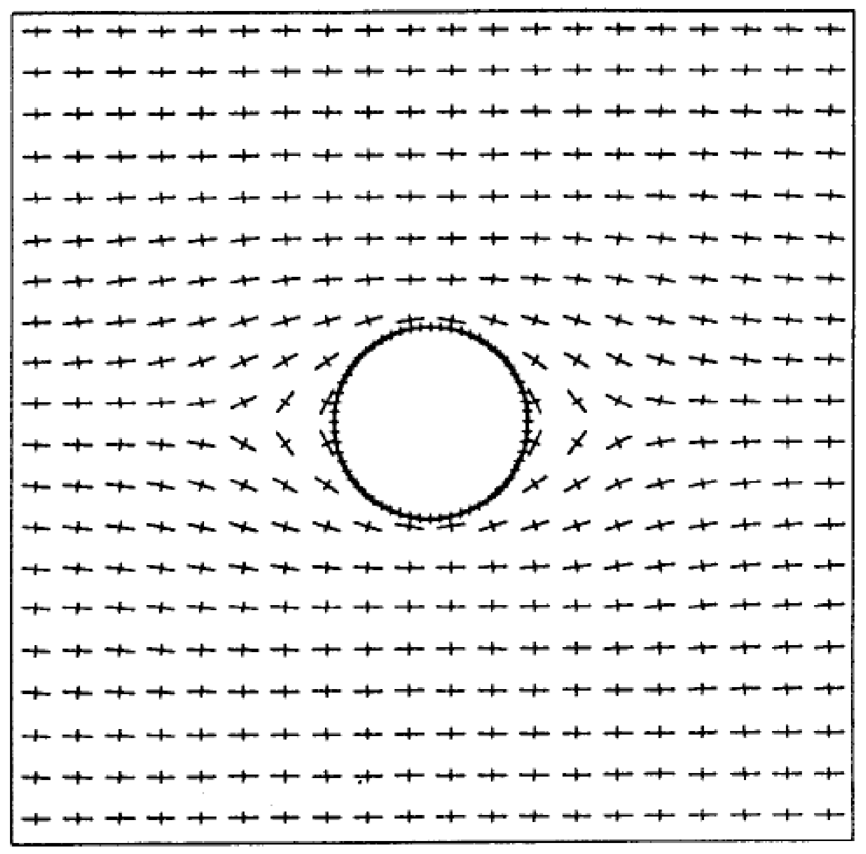
Induced stresses: Closed form solutions

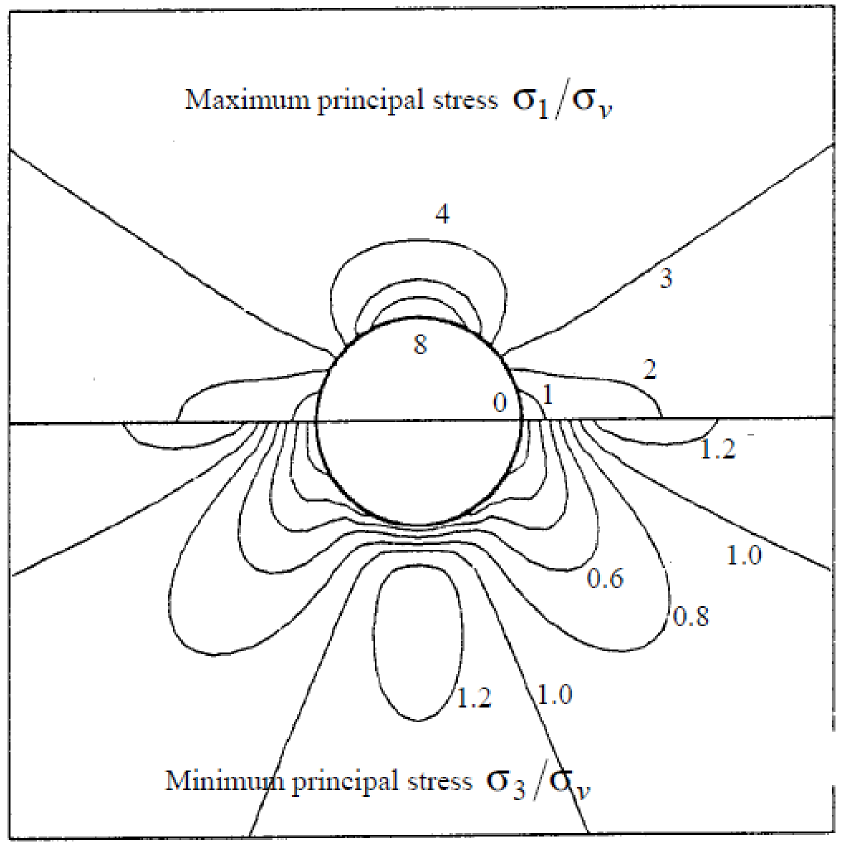
- When an underground opening is excavated into a stressed rock mass, the stresses in the vicinity of the new opening are re-distributed.
- Before the tunnel is excavated, the in situ stresses σ_v, σ_{h1}, and σ_{h2} are uniformly distributed in the slice of rock under consideration.
- After removal of the rock from within the tunnel, the stresses in the immediate vicinity of the tunnel are changed and new stresses are induced.

Induced stresses: Closed form solutions



Induced stresses: Closed form solutions





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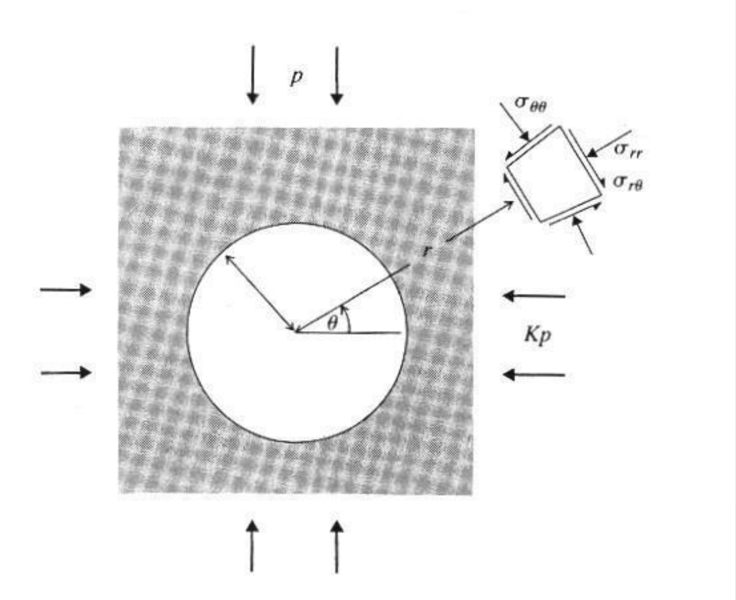
Induced stresses: Closed form solutions

- The convention used in rock mechanics is that **compressive stresses are always positive** and the three principal stresses are numbered such that σ_1 is the largest and σ_3 is the smallest (algebraically) of the three.
- An analytical solution for the stress distribution in a stressed elastic plate containing a circular hole was published by **Kirsch (1898)** and this formed the basis for many early studies of rock behaviour around tunnels and shafts.

Circular tunnel

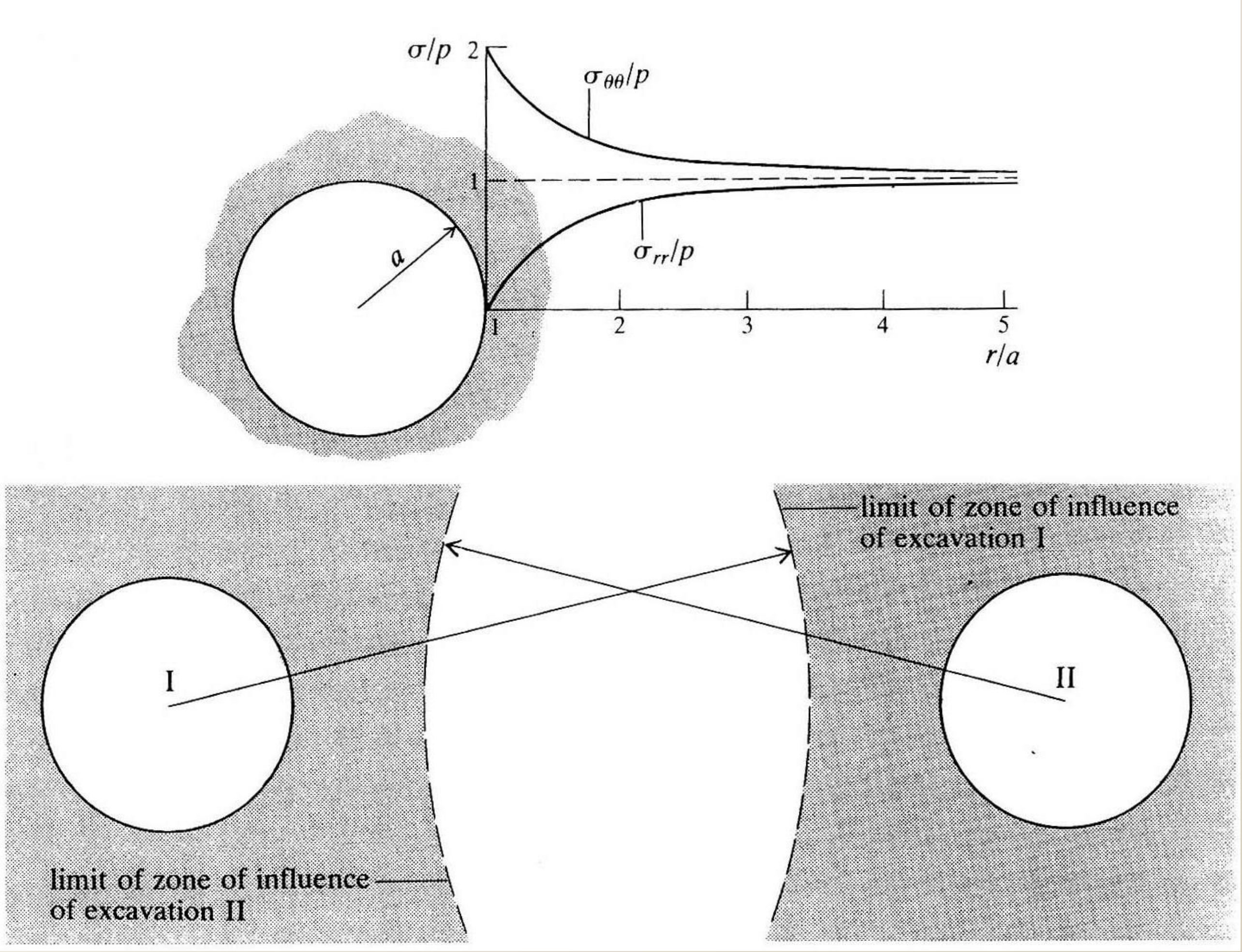


$$\sigma_{rr} = \frac{p}{2}\left[(1+K)\left(1-\frac{R^2}{r^2}\right)-(1-K)\left(1-4\frac{R^2}{r^2}+3\frac{R^4}{r^4}\right)\cos 2\theta\right]$$

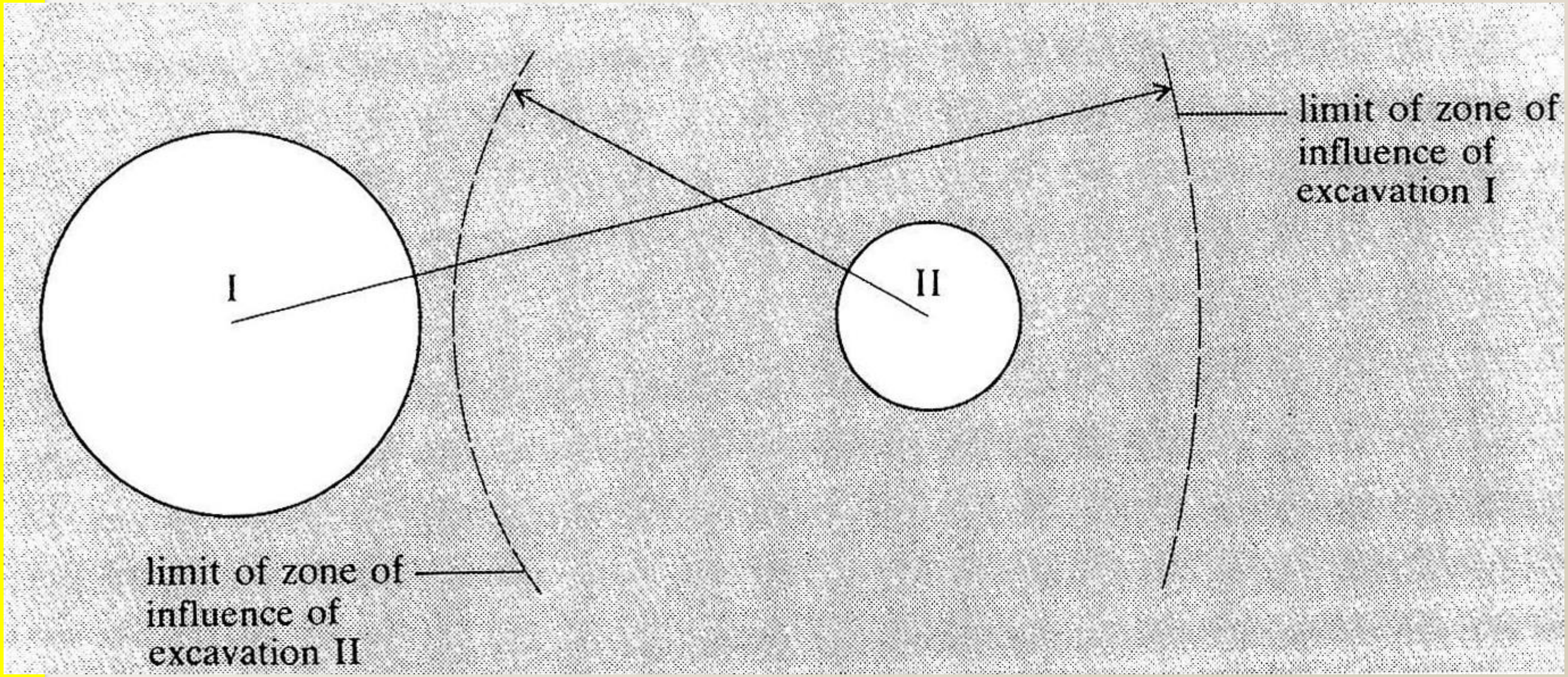
$$\sigma_{\theta\theta} = \frac{p}{2}\left[(1+K)\left(1+\frac{R^2}{r^2}\right)+(1-K)\left(1+3\frac{R^4}{r^4}\right)\cos 2\theta\right]$$

$$\sigma_{r\theta} = \frac{p}{2}\left[(1-K)\left(1+2\frac{R^2}{r^2}-3\frac{R^4}{r^4}\right)\sin 2\theta\right]$$

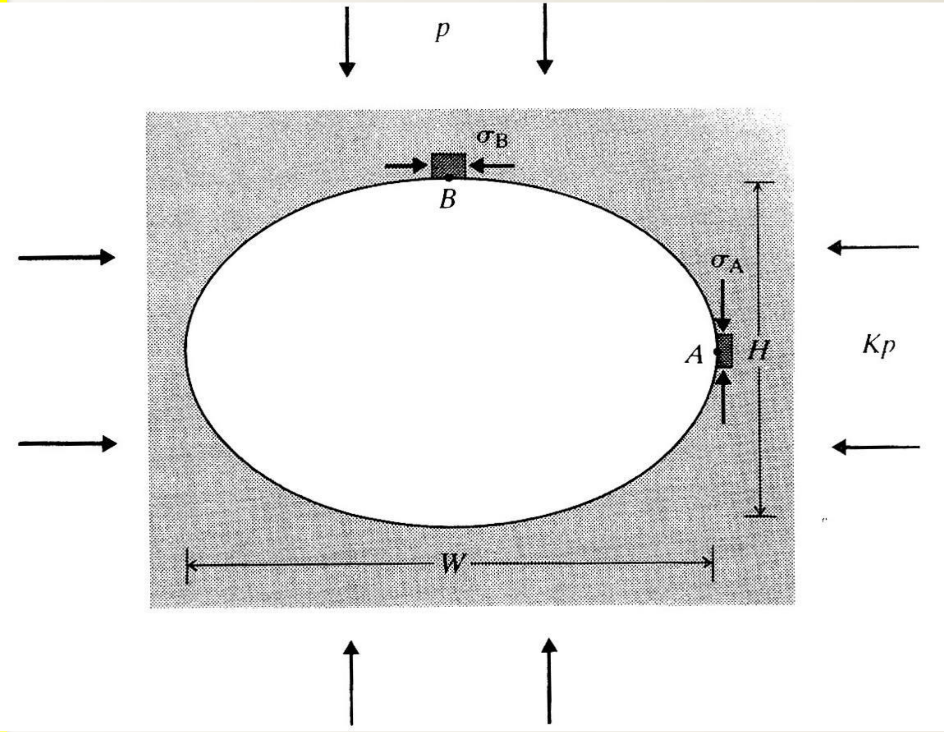
Circular tunnel: stress zone of influence



Circular tunnel: stress zone of influence



Ellipse tunnel

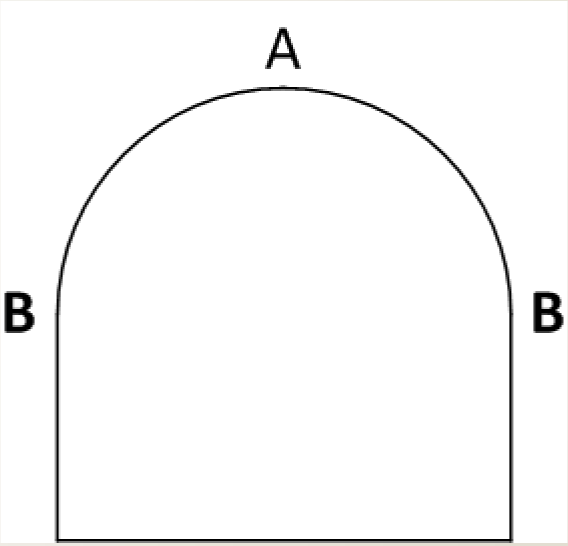


$$\sigma_A = p\left(1 - K + 2q\right)$$

$$\sigma_B = p\left(K - 1 + \frac{2K}{q}\right)$$

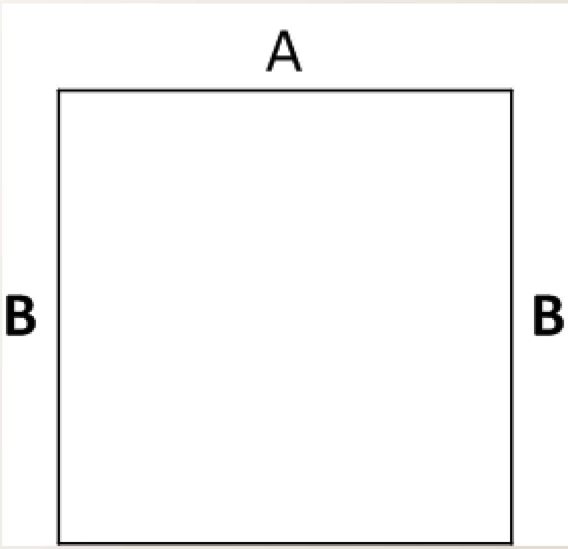
$$q = \frac{W}{H}$$

Horse shoe tunnel



- $\sigma_h = \sigma_v$
  - $\sigma_A = 2.2\ \sigma_v$
  - $\sigma_B = 1.3\ \sigma_v$
- $\sigma_h = 0.5\ \sigma_v$
  - $\sigma_A = 0.6\ \sigma_v$
  - $\sigma_B = 1.8\ \sigma_v$
- $\sigma_h = 0.33\ \sigma_v$
  - $\sigma_A = 0.1\ \sigma_v$
  - $\sigma_B = 1.9\ \sigma_v$

Square tunnel



- $\sigma_h = \sigma_v$
  - $\sigma_A = 1.1\ \sigma_v$
  - $\sigma_B = 1.1\ \sigma_v$
- $\sigma_h = 0.5\ \sigma_v$
  - $\sigma_A = 0.1\ \sigma_v$
  - $\sigma_B = 1.6\ \sigma_v$
- $\sigma_h = 0.33\ \sigma_v$
  - $\sigma_A = -0.3\ \sigma_v$
  - $\sigma_B = 1.8\ \sigma_v$

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Induced stresses: Closed form solutions

- Closed form solutions still possess great value for conceptual understanding of behaviour and for the testing and calibration of numerical models.
- For design purposes, however, these models are restricted to very simple geometries and material models. They are of limited practical value.
- Fortunately, with the development of computers, many powerful programs which provide numerical solutions to these problems are now readily available.

LESSON 6 AND 7. INDUCED STRESSES: NUMERICAL METHODS; DISCONTINUITIES & INDUCED STRESSES

Induced stresses: Numerical methods

- Most underground excavations are irregular in shape and are frequently grouped close to other excavations.
- These groups of excavations can form a set of complex three dimensional shapes.
- In addition, because of the presence of geological features such as faults and intrusions, the rock properties are seldom uniform within the rock volume of interest.

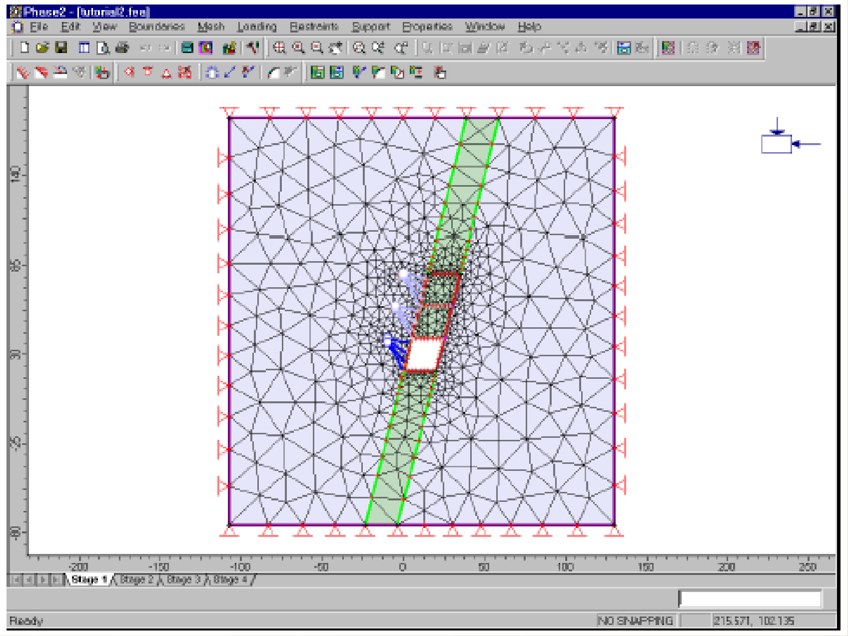
Induced stresses: Numerical methods

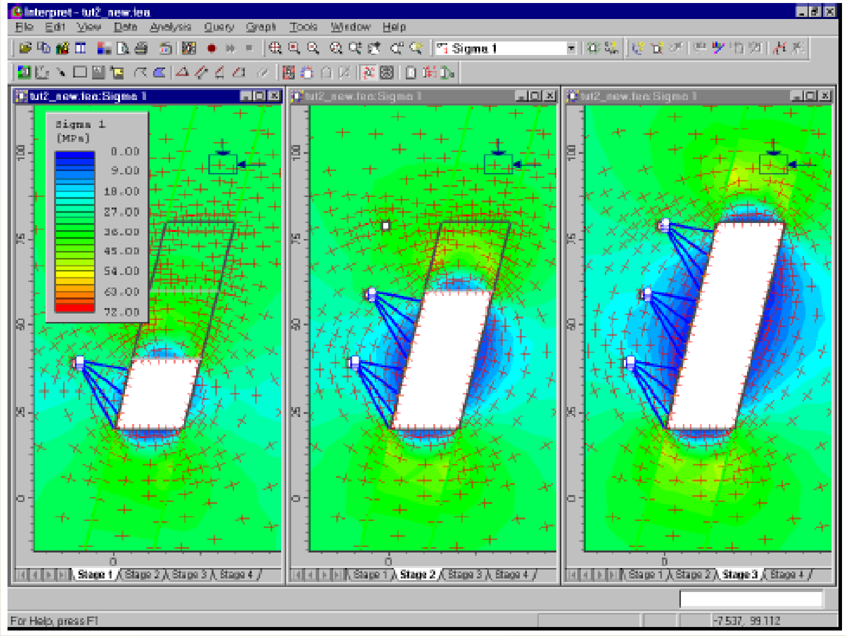
- Consequently, the closed form solutions described earlier are of limited value in calculating the stresses, displacements and failure of the rock mass surrounding underground excavations.
- Fortunately a number of computer-based numerical methods have been developed over the past few decades and these methods provide the means for obtaining **approximate solutions** to these problems.

Induced stresses: Numerical methods

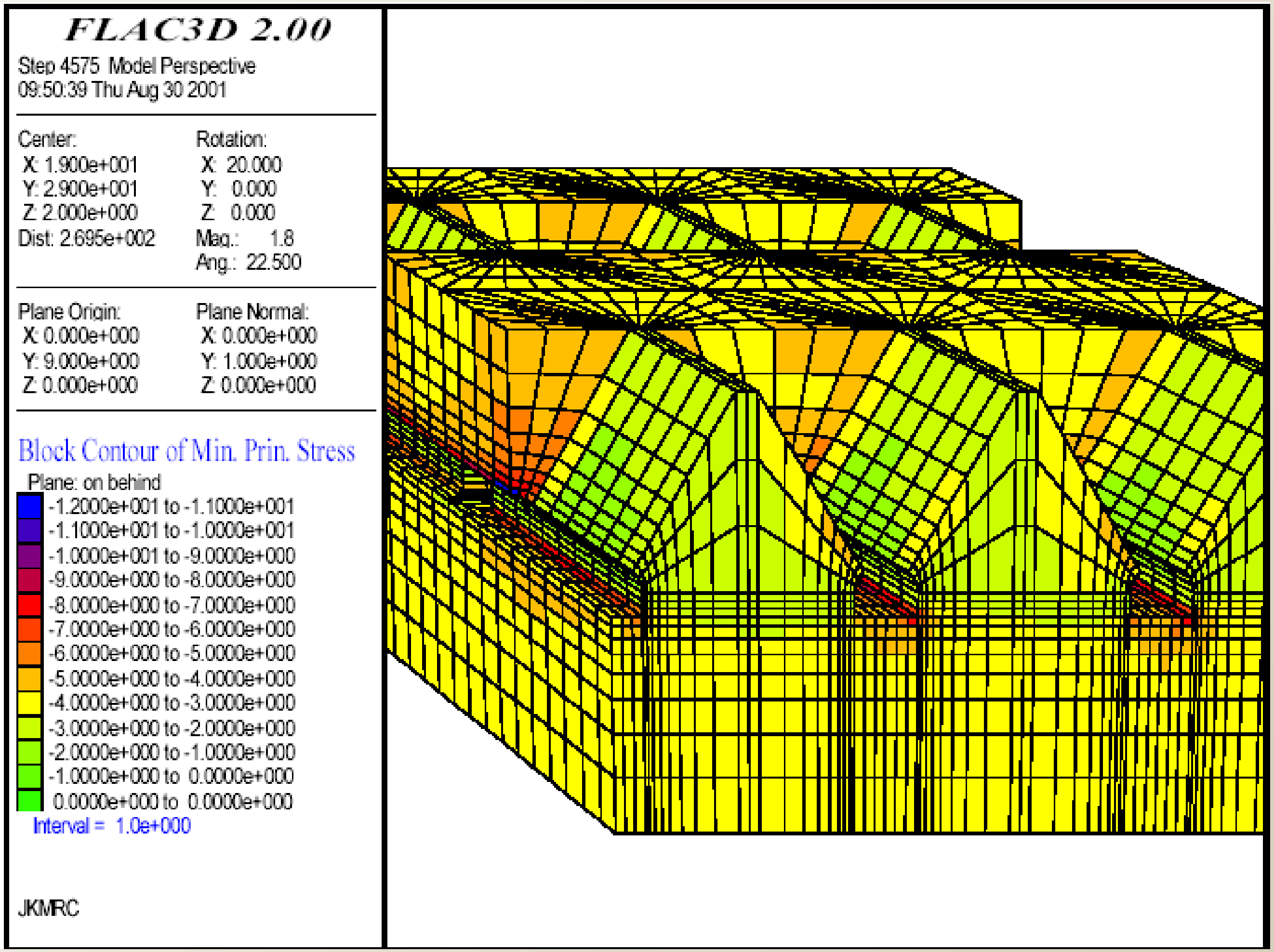
- **Boundary methods**, in which only the boundary of the excavation is divided into elements and the interior of the rock mass is represented mathematically as an infinite continuum.
- **Domain methods**, in which the interior of the rock mass is divided into geometrically simple elements each with assumed properties. The collective behaviour and interaction of these simplified elements model the more complex overall behaviour of the rock mass.

Induced stresses: Numerical methods





Induced stresses: Numerical methods

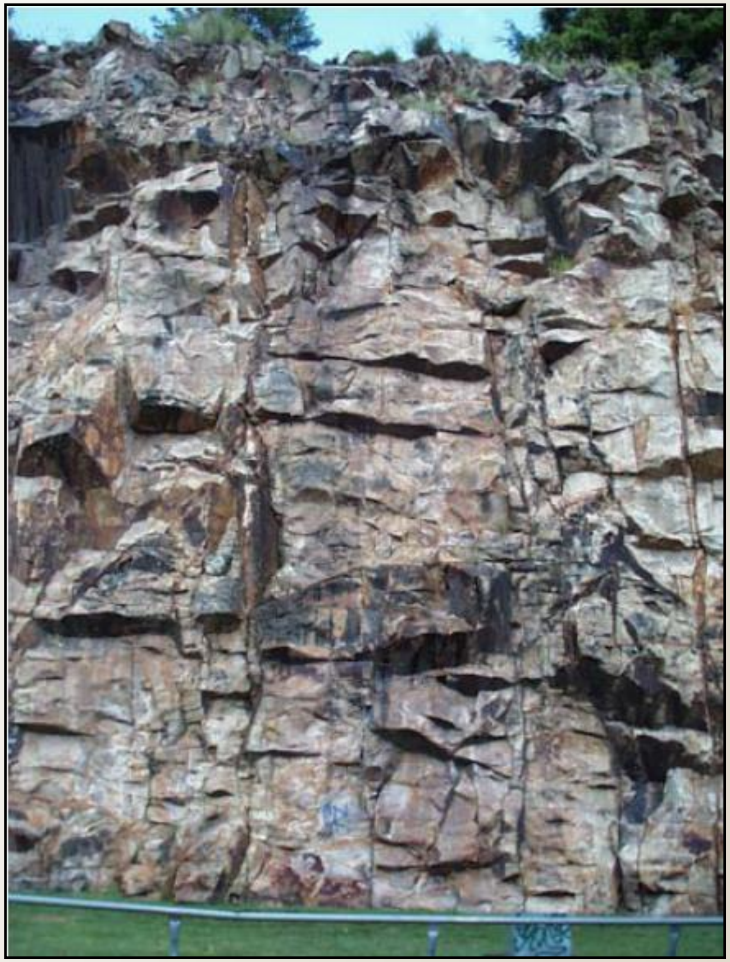


LESSON 7. DISCONTINUITIES & INDUCED STRESSES

Rock mass modelling

- **CHILE**
  Continuous, **H**omogeneous, **I**sotropic, and **L**inearly **E**lastic
- **DIANE**
  Discontinuous, **I**nhomogeneous, **A**nisotropic, and **N**ot-**E**lastic
- These refer to two ways of thinking about and modelling the rock mass.

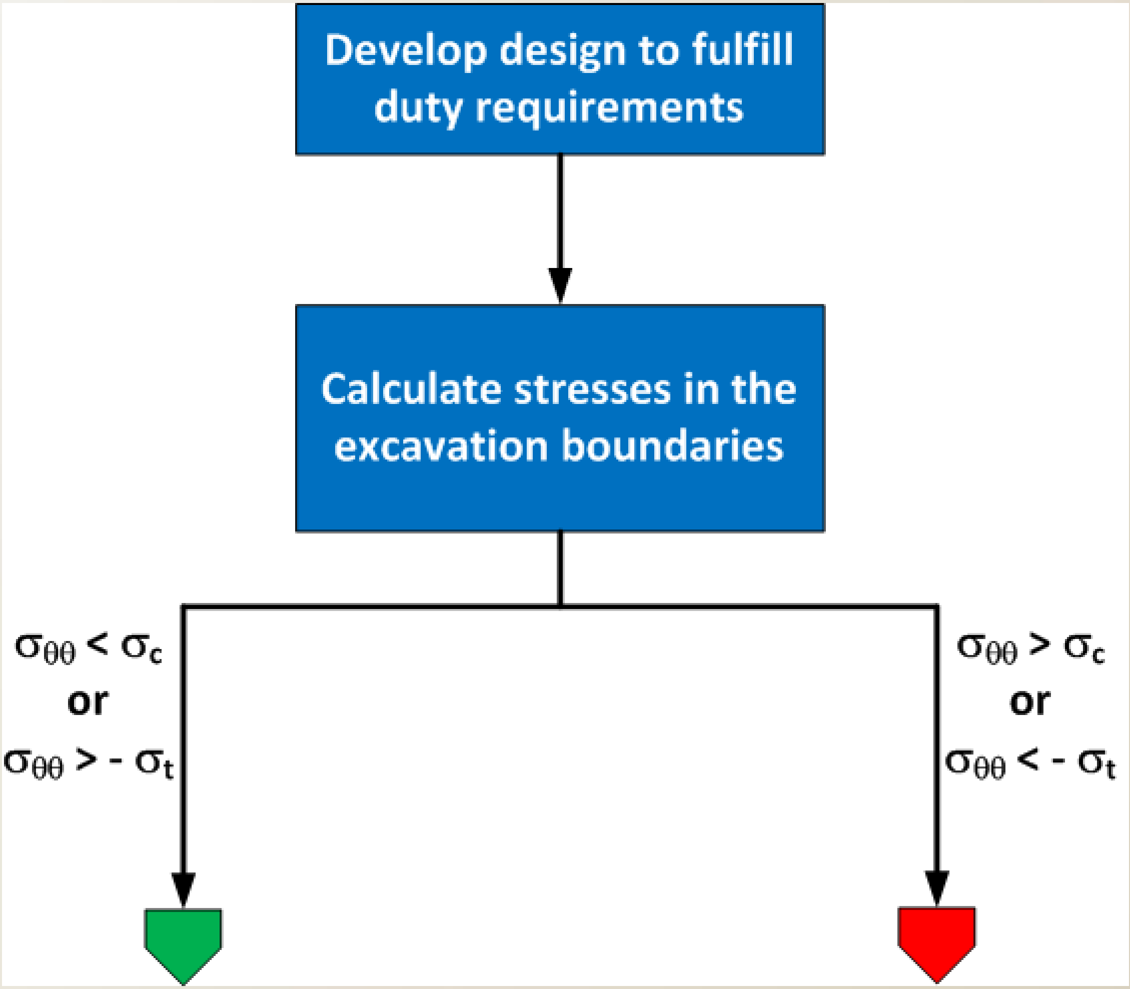
Rock mass modelling



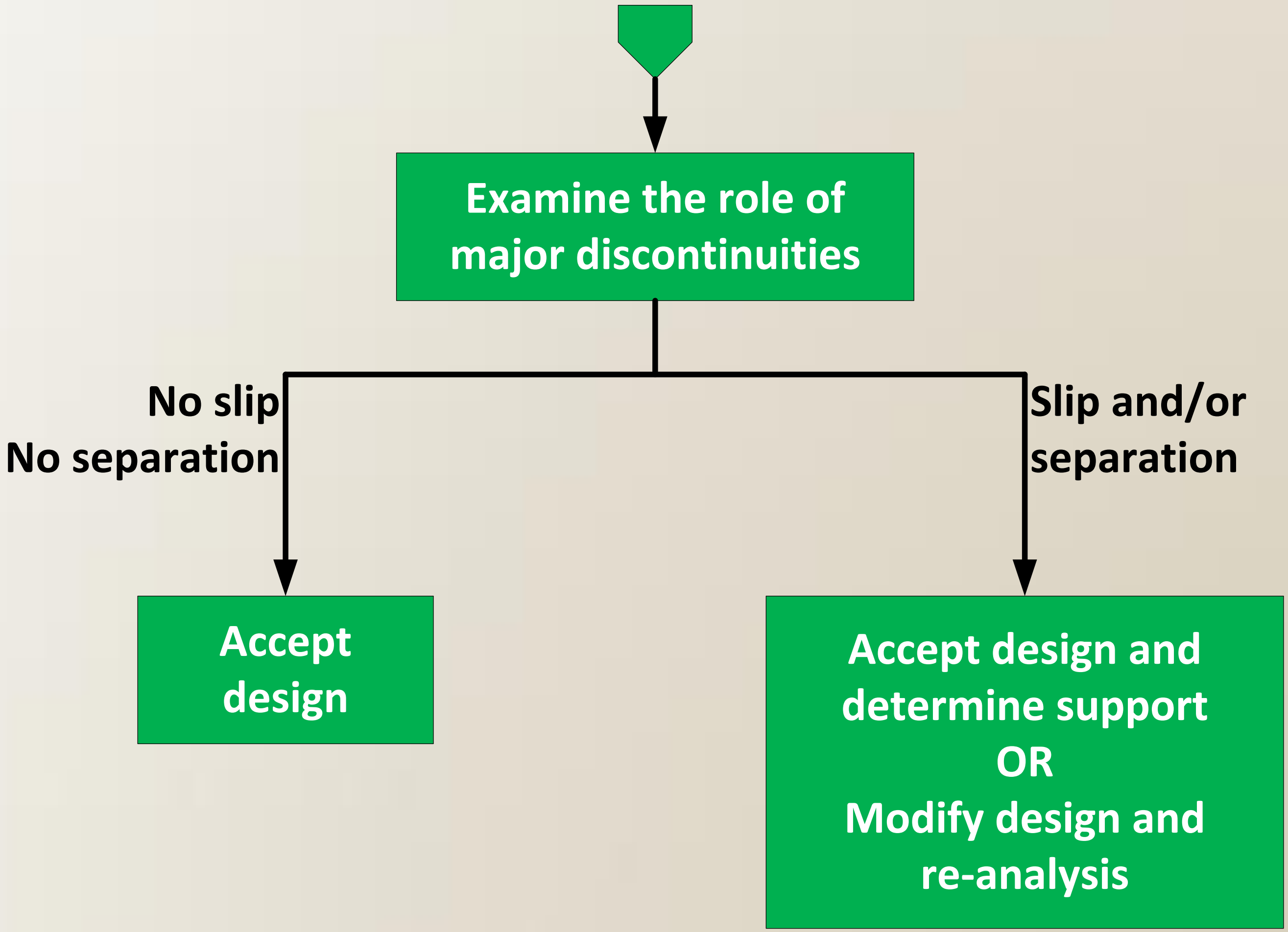
Rock mass modelling

- In the CHILE case, we assume an ideal type of material which is not fractured, or if it is fractured the fracturing can be incorporated in the elastic continuum properties.
- In the DIANE case, the nature of the real rock mass is recognised and we model accordingly, still often making gross approximations.
- Rock mechanics started with the CHILE approach and has now developed techniques to enable the DIANE approach to be implemented.

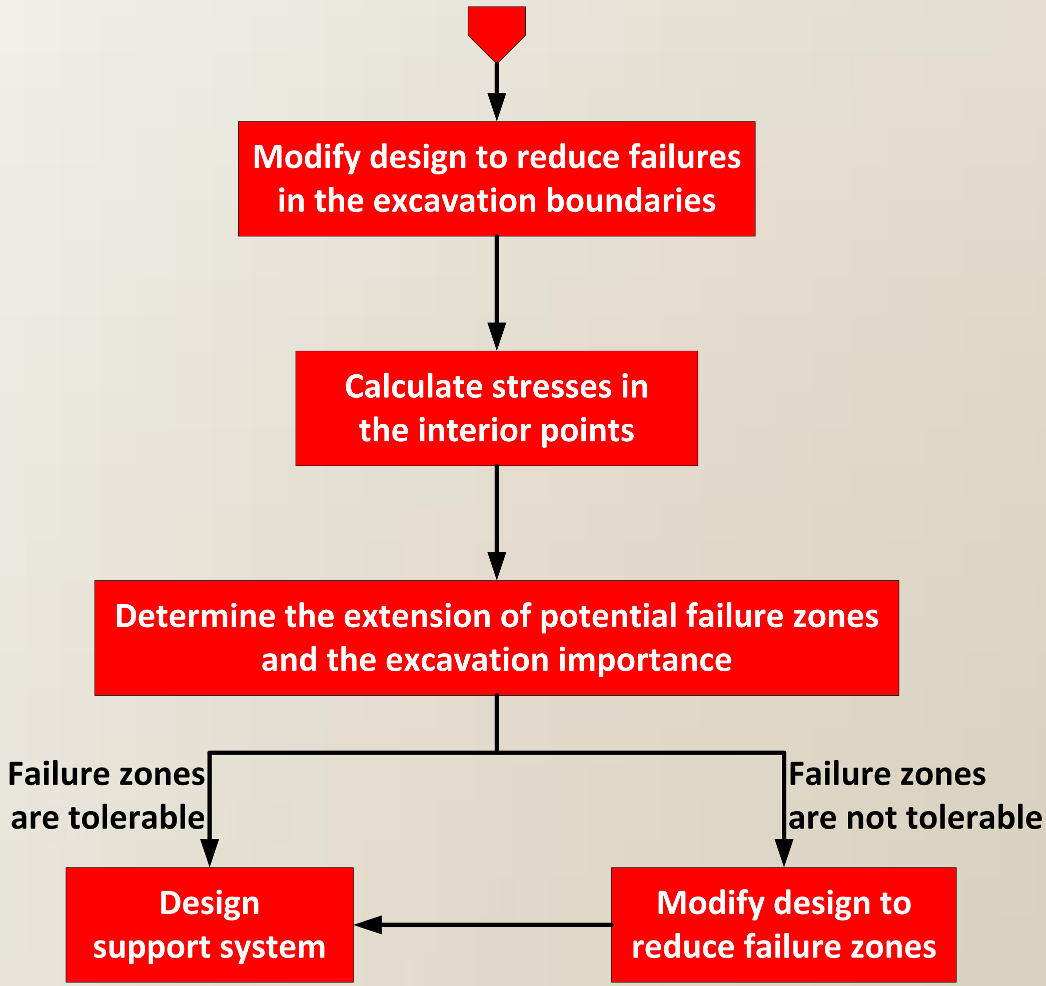
Excavation in massive elastic rock mass: Design methodology



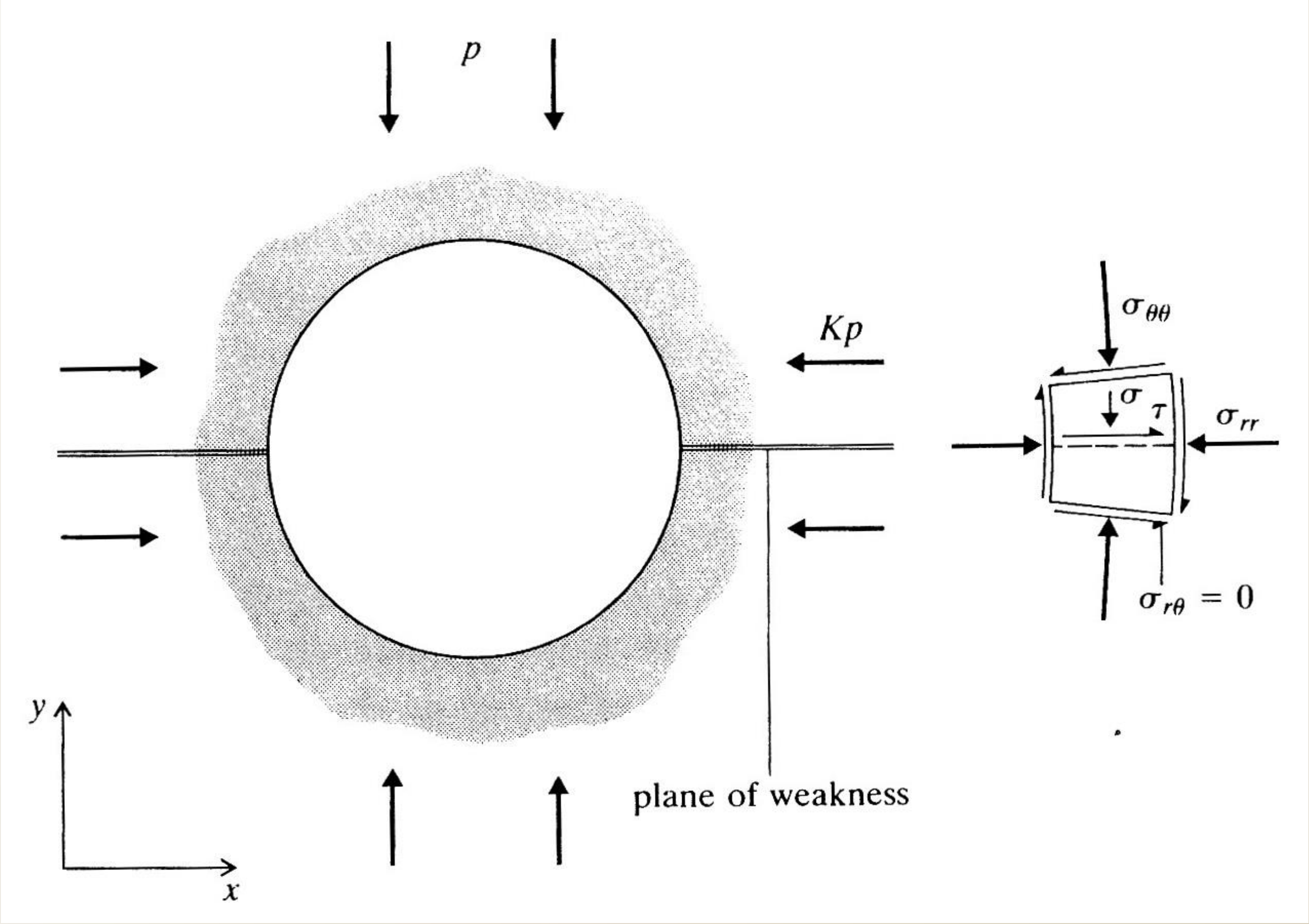
Excavation in massive elastic rock mass: Design methodology



Excavation in massive elastic rock mass: Design methodology



Case 1



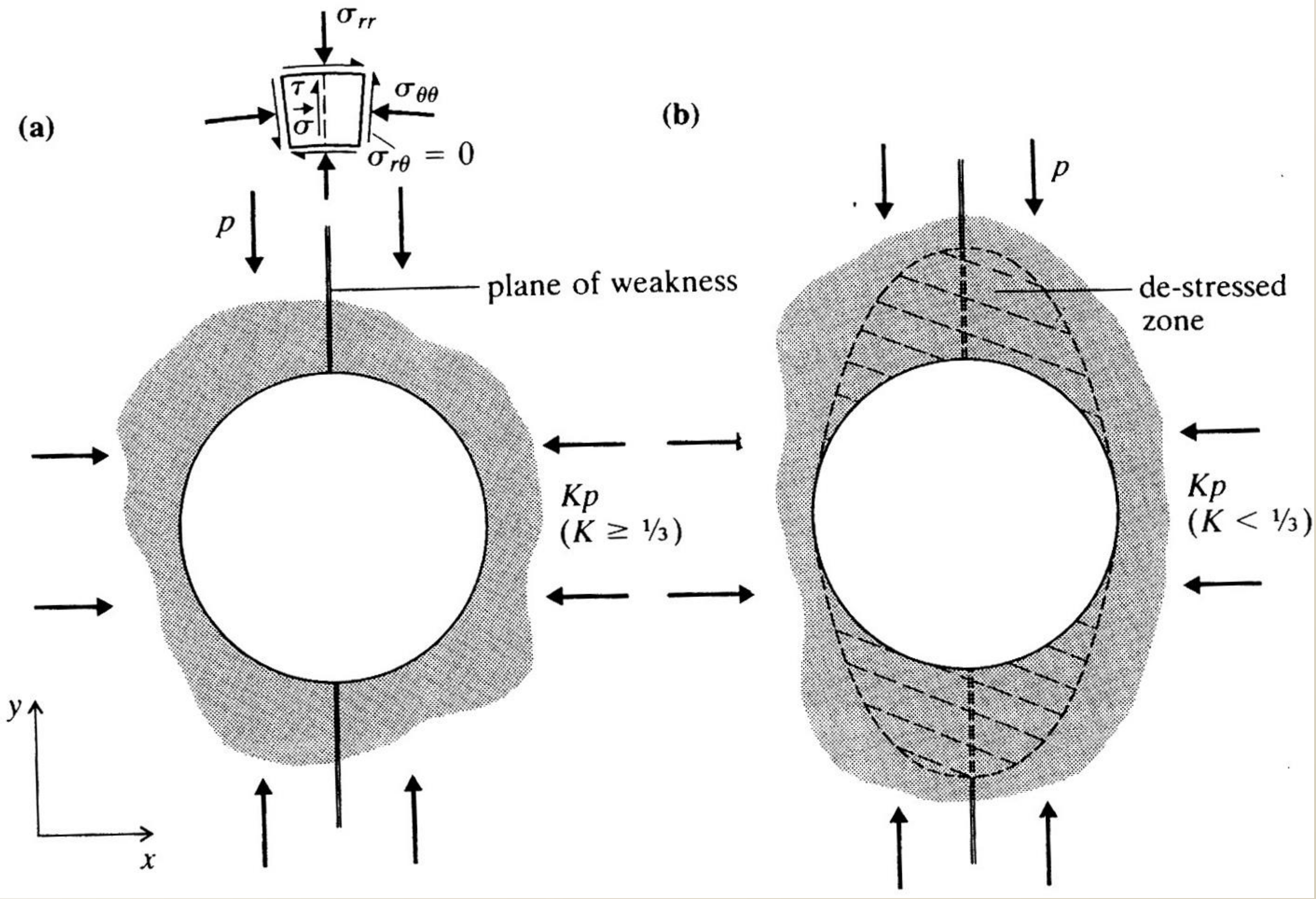
Case 1

- Using Kirsch equation for $\theta = 0$:

  $\sigma_{r\theta} = 0$ for all r

  $\sigma_{\theta\theta}$ and σ_{rr} are the principal stresses.
- Shear stress on the weak plane is zero → no slip
- The weak plane does not affect the elastic stress distribution.

Case 2



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Case 2

- Using Kirsch equation for $\theta = 90$:

  $\sigma_{r\theta} = 0$ for all r

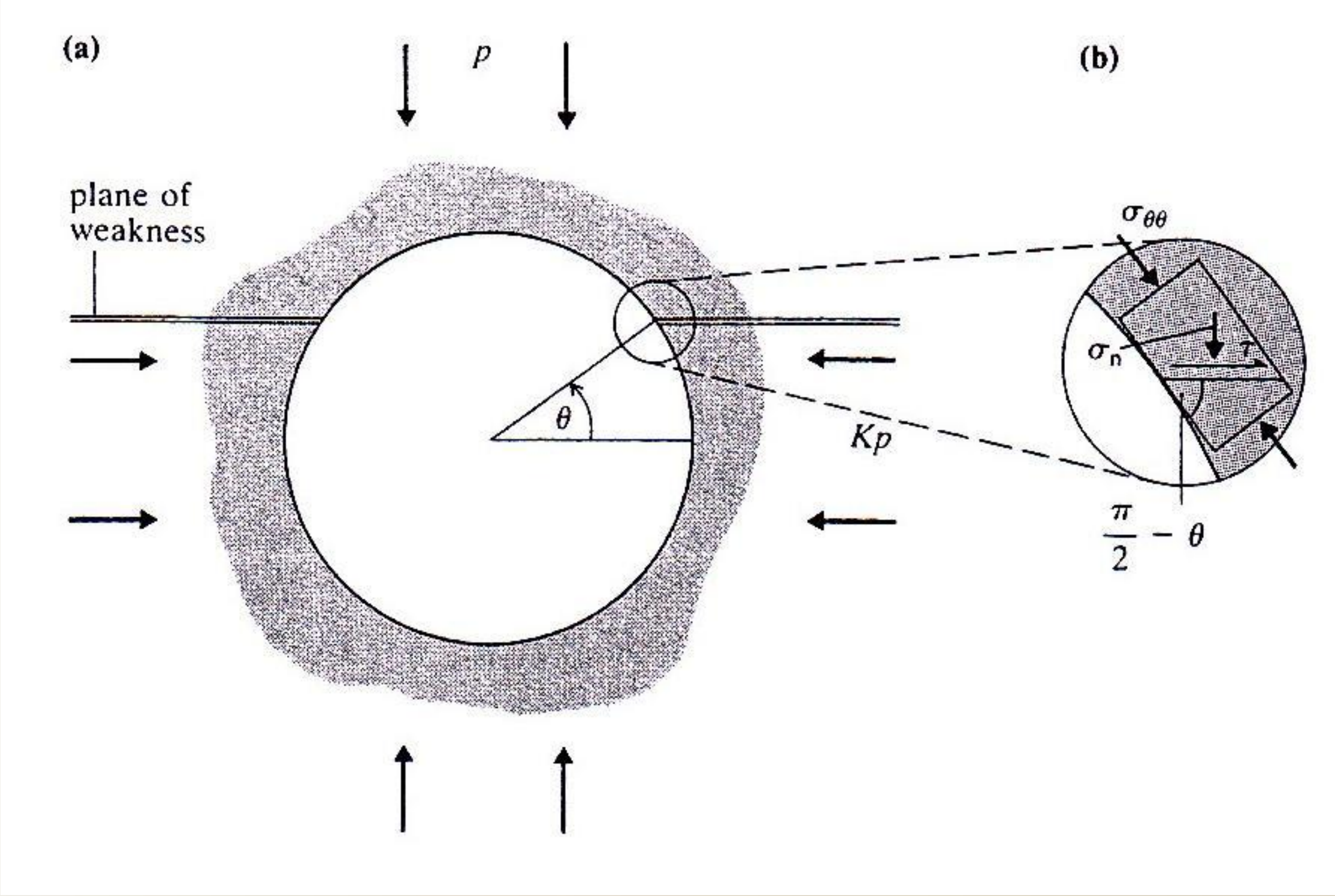
- Shear stress on the weak plane is zero → no slip
- In the roof (r = R):

$$\sigma_{rr} = 0$$

$$\sigma_{\theta\theta} = p(3K - 1)$$

$K < 1/3$ → $\sigma_{\theta\theta} < 0$ → Tension stress → Separation

Case 3



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Case 3

Using **simple** trigonometric analysis:

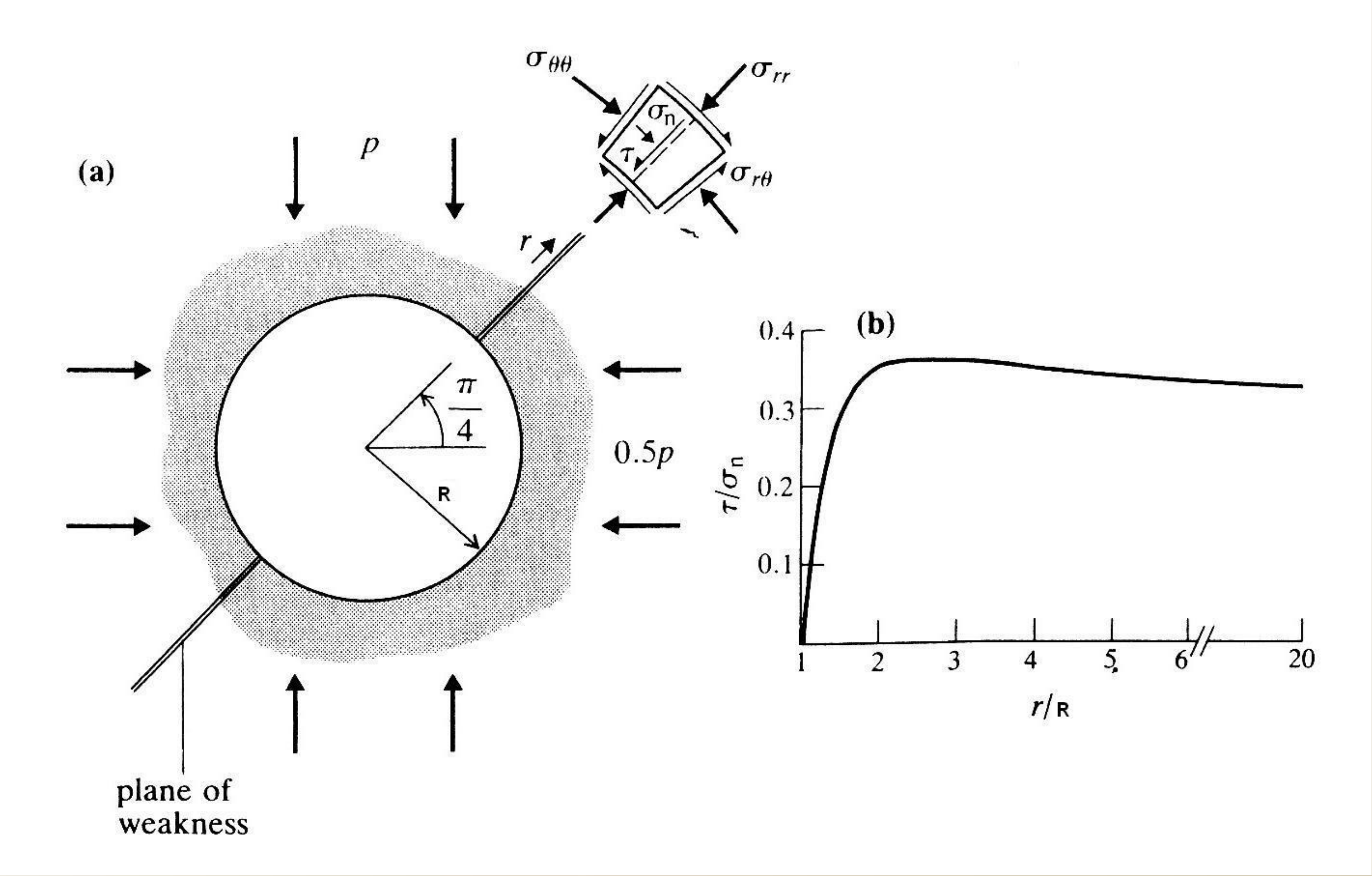
- $\sigma_n = \sigma_{\theta\theta} \cos \theta$
- $\tau = \sigma_{\theta\theta} \sin \theta$

Based on **Mohr-Coulomb failure criterion** assuming

C = 0 for weak plane → $\boldsymbol{\tau = \sigma_n \tan \phi}$:

- $\sigma_{\theta\theta} \sin \theta = \sigma_{\theta\theta} \cos \theta \tan \phi \rightarrow \sigma_{\theta\theta} \tan \theta = \sigma_{\theta\theta} \tan \phi$
- Limit condition for slip: $\boldsymbol{\theta = \phi}$

Case 3



(a)
p
$\sigma_{\theta\theta}$
σ_{rr}
σ_n
τ
$\sigma_{r\theta}$
r
$\pi/4$
R
0.5p
plane of weakness
(b)
0.4
0.3
0.2
0.1
τ/σ_n
1
2
3
4
5
6
20
r/R

Case 4

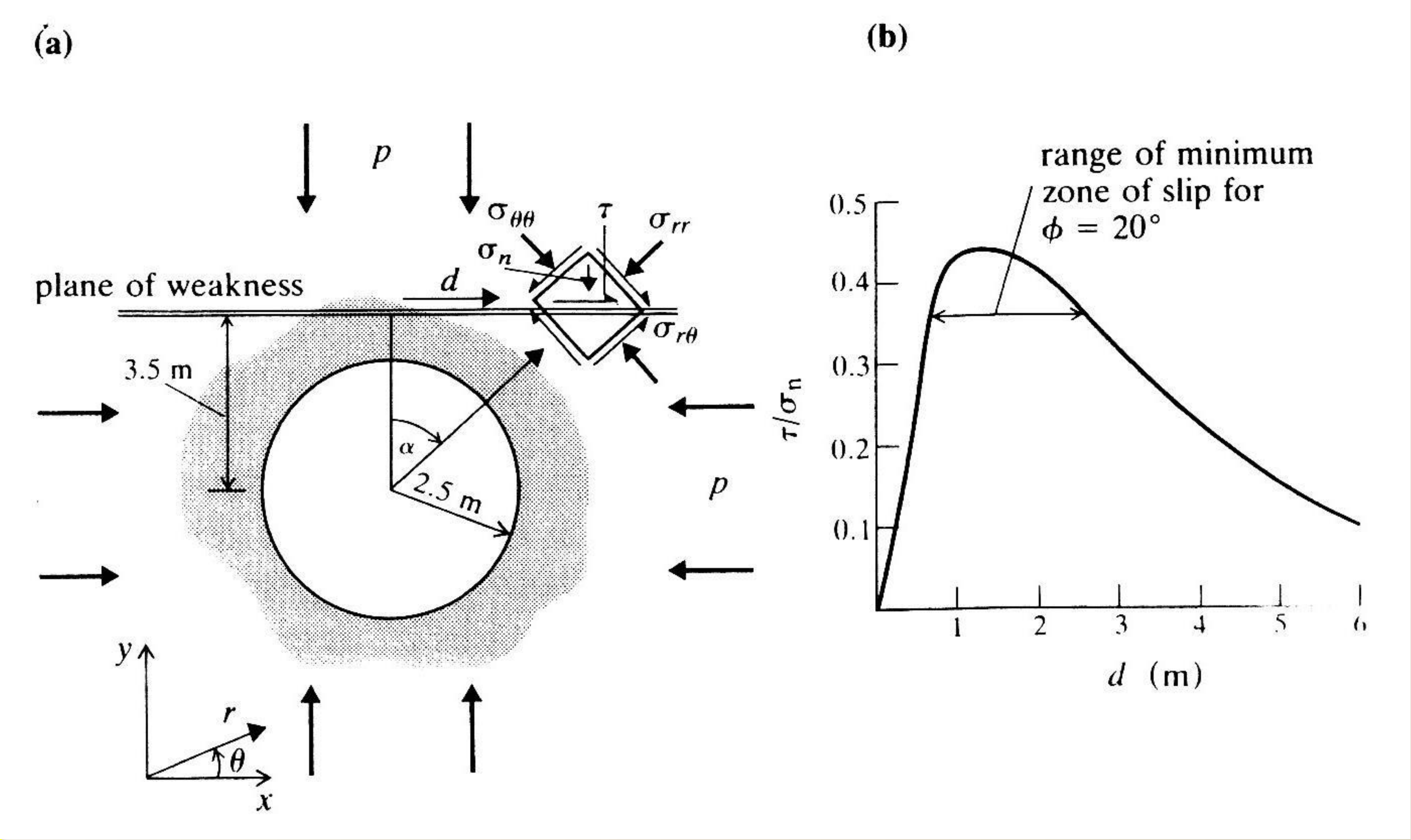
Using Kirsch equation for $\boldsymbol{\sigma_{\theta\theta}}$ **(=**$\boldsymbol{\sigma_n}$**)** and $\boldsymbol{\sigma_{r\theta}}$ **(=**$\boldsymbol{\tau}$**)** with $\theta = 45^O$ and K = 0.5:

$$\sigma_n = 0.75p\left(1 + \frac{R^2}{r^2}\right)$$

$$\tau = 0.25p\left(1 + 2\frac{R^2}{r^2} - 3\frac{R^4}{r^4}\right)$$

r/R ratio that gives **maximum** $\boldsymbol{\tau/\sigma_n}$ **ratio** can be found for a particular ϕ → see (b) for $\boldsymbol{\phi = 19.6^O}$

Case 5



(a)
p
$\sigma_{\theta\theta}$
τ
σ_{rr}
σ_n
plane of weakness
d
$\sigma_{r\theta}$
3.5 m
α
2.5 m
p
y
r
θ
x
(b)
range of minimum
zone of slip for
$\phi = 20°$
0.5
0.4
0.3
0.2
0.1
τ/σ_n
1
2
3
4
5
6
d (m)

Case 5

Using Kirsch equation with K = 1.0 and **simple** trigonometric analysis:

$$\sigma_n = p\left(1 - \frac{R^2}{r^2}\cos 2\alpha\right)$$

$$\tau = p\frac{R^2}{r^2}\sin 2\alpha$$

d that gives **maximum τ/σ_n ratio** can be found for a particular ϕ →see (b) for **ϕ = 20^O**