NATIONAL TECHNICAL UNIVERSITY OF ATHENS

School of Civil Engineering – Geotechnical Department

Computational Methods in the Analysis of Underground Structures

Spring Term 2023 – 24

Lecture Series in Postgraduate Programs:

1. Analysis and Design of Structures (DSAK)
2. Design and Construction of Underground Structures (SKYE)

Instructor: Michael Kavvadas, Emer. Professor NTUA

LECTURE 1: Introduction

20.02.2024

Objectives of tunnelling: Mountain (deep) and Urban (shallow) tunnels

1. Mountain tunnels – usually deep tunnels (d > 30m)

- Excavation and support of a stable tunnel face and section, which fulfills operational requirements, and is safe for the tunnelling personnel → no collapse
- Minimisation of support: Relatively large radial wall convergence and axial face extrusion are not critical. They are desirable, since they mobilise the surrounding rockmass → radial stress reduction (increased deconfinement) → arch development → reduced support.
  Note: Excessive deformations can degrade the surrounding rockmass, causing structural (wedge) failures and increasing support requirements.

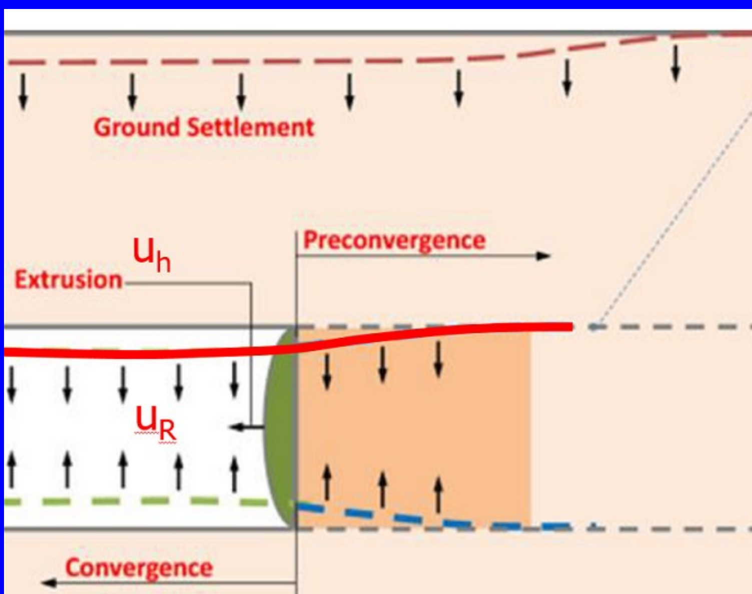
Methods of excavation and support:
(1) Conventional excavation (NATM / SCL), with two-stage support (immediate and final)
(2) Mechanised excavation (TBM), with two-stage or one-stage support (precast elements)

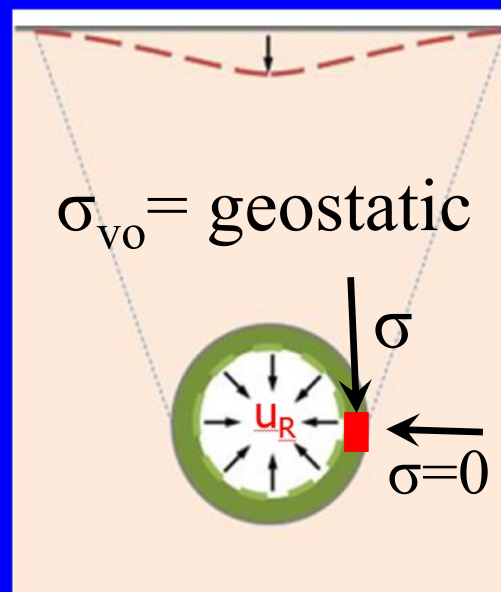
Objectives of tunnelling: Mountain (deep) and Urban (shallow) tunnels

2. Urban tunnels – usually shallow tunnels (d < 30m)

- Excavation and support of a stable tunnel face and section, which fulfills operational requirements, and is safe for the tunnelling personnel → no collapse
- Minimization of tunnel wall convergence and face extrusion, in order to minimize ground deformations and surface settlement → avoid damage of surface structures.

  However: Small deformations → small deconfinement → small radial stress reduction → tunnel support loads almost equal to the geostatic. This is not a serious problem, because the geostatic loads are small (shallow tunnel), e.g. depth = 30m → $\sigma_{vo} = \gamma\ d = 20 \times 30 = 600$ kPa





Hoop stress in lining (thickness t = 0.35m, tunnel radius R = 4.5m):
$\sigma = \sigma_{vo}$ (R/t) = 0.6 x (4.5/0.35) = 7.7 MPa, reasonable for concrete with strength:
f_{ck} = 30 MPa

Methods of excavation and support:

(1) Mechanised excavation (TBM) and final support with precast elements

(2) Conventional excavation (NATM), with very stiff immediate support (+ face reinforcement) and final support

Objectives of tunnelling: Mountain (deep) and Urban (shallow) tunnels

2. Urban tunnels – usually shallow tunnels (d < 30m)

Catastrophic failure (2003) along Douk. Plakentias Av. during conventional tunnelling of the Athens Metro tunnels

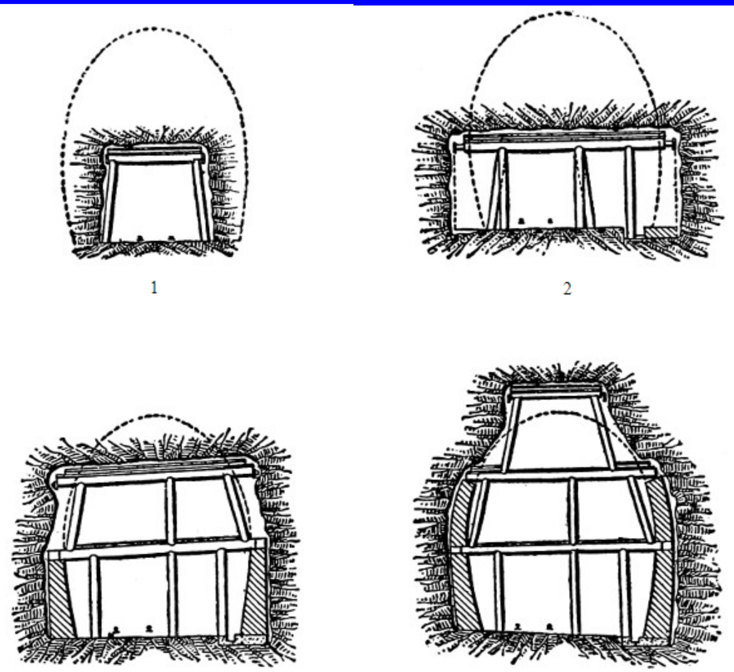
Tunnel excavation in good ground

Tunnel excavation (using explosives) in very good ground. Hardly any support is required - Hydroelectric tunnel in the Himalayas (India)

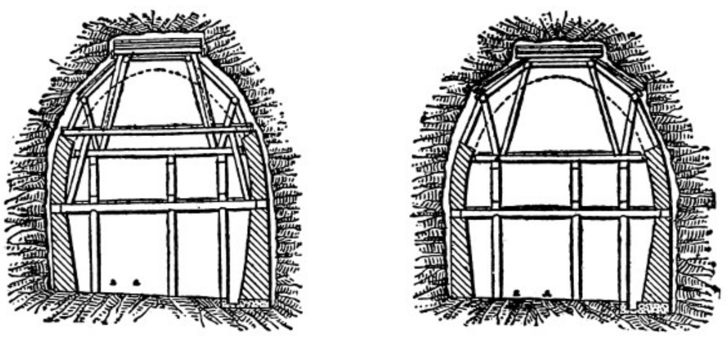
Tunnel excavation (using explosives) in good ground. Support with sporadic rockbolts (Olympiada gold mine, Greece)

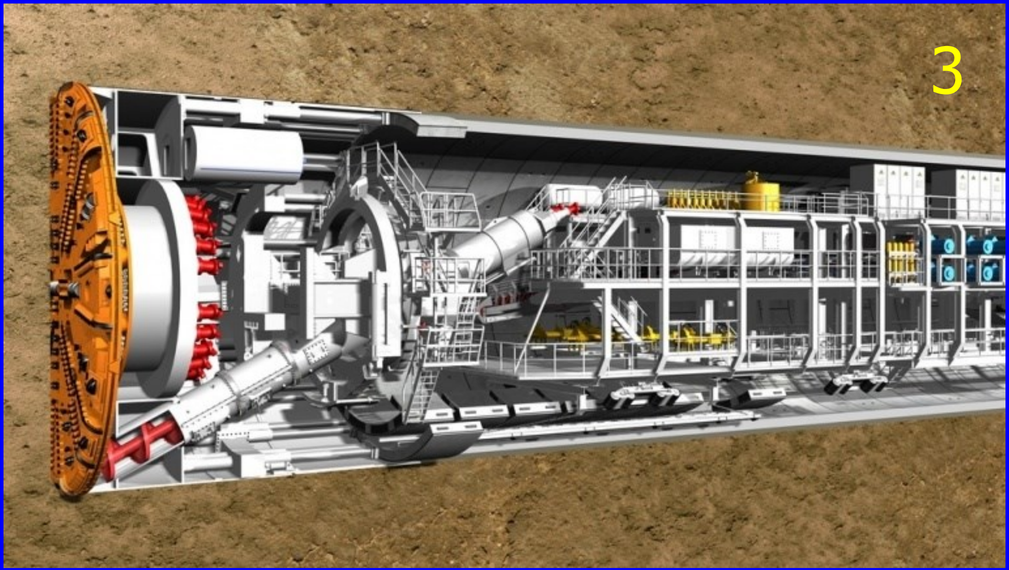
Tunnel excavation methods

1. Old methods with wooden/steel support and multiple phases
2. Conventional excavation (NATM)
3. Mechanised excavation (TBM)

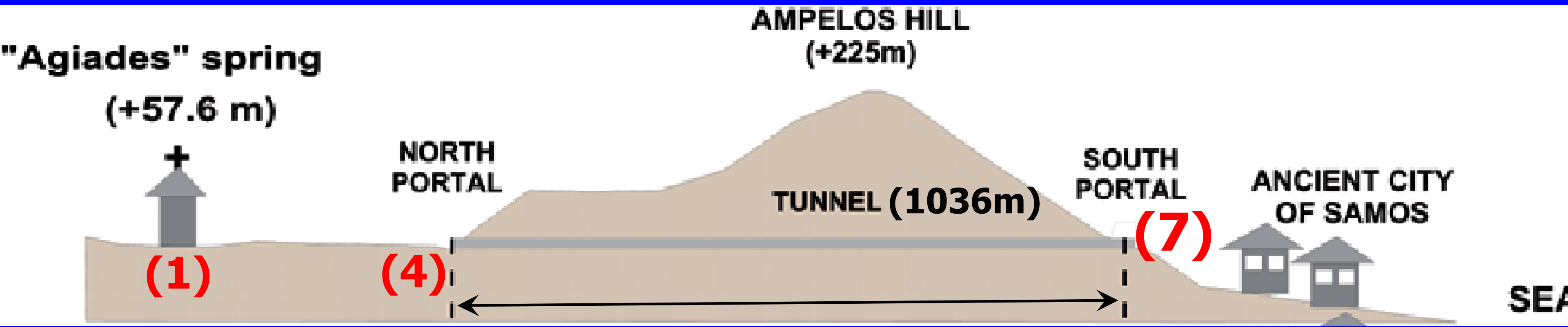


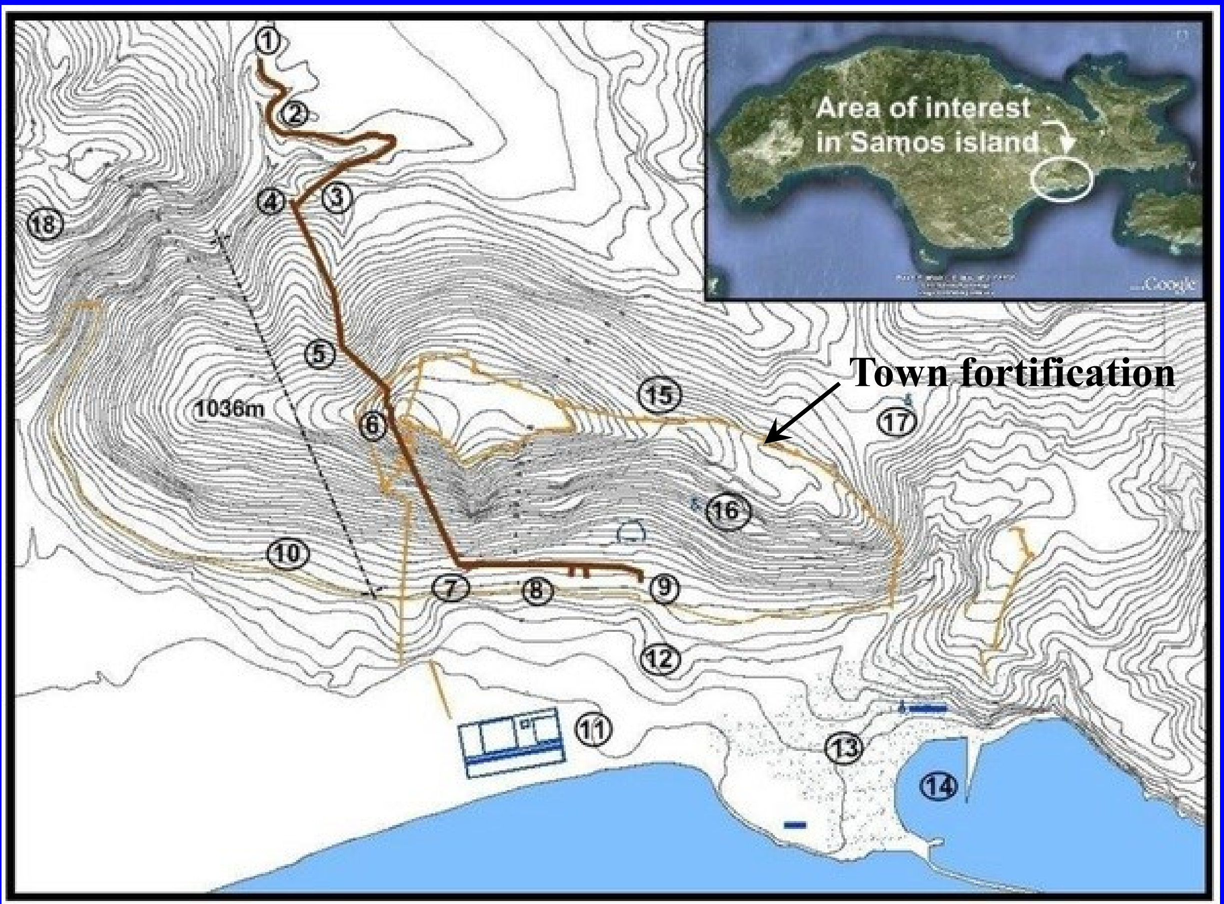
1 → Simplon tunnel (1912-21)





Tunnel excavation methods – Eupalinus Tunnel in Samos (Greece)





The tunnel was dug in the middle of the sixth century BCE, in order to supply the ancient capital of Samos (today called Pythagoreion) with fresh water.

It was dug by two groups (one from each end) working under the direction of the engineer Eupalinos from Megara.

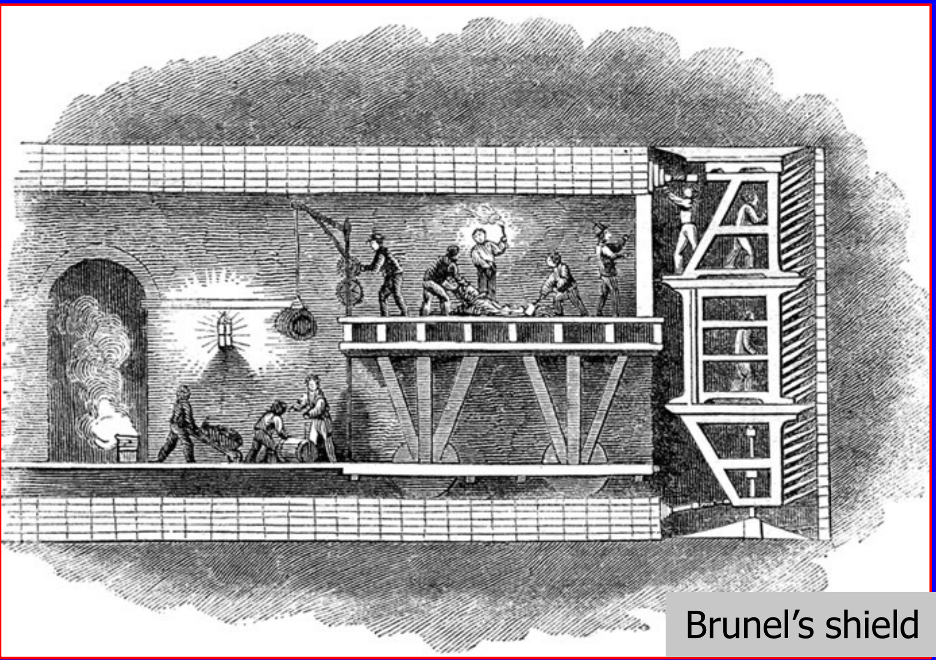
Tunnel excavation methods - Tunnel of Eupalinus in Samos (Greece)

Tunnel excavation methods
Old methods with wooden/steel support and multiple phases

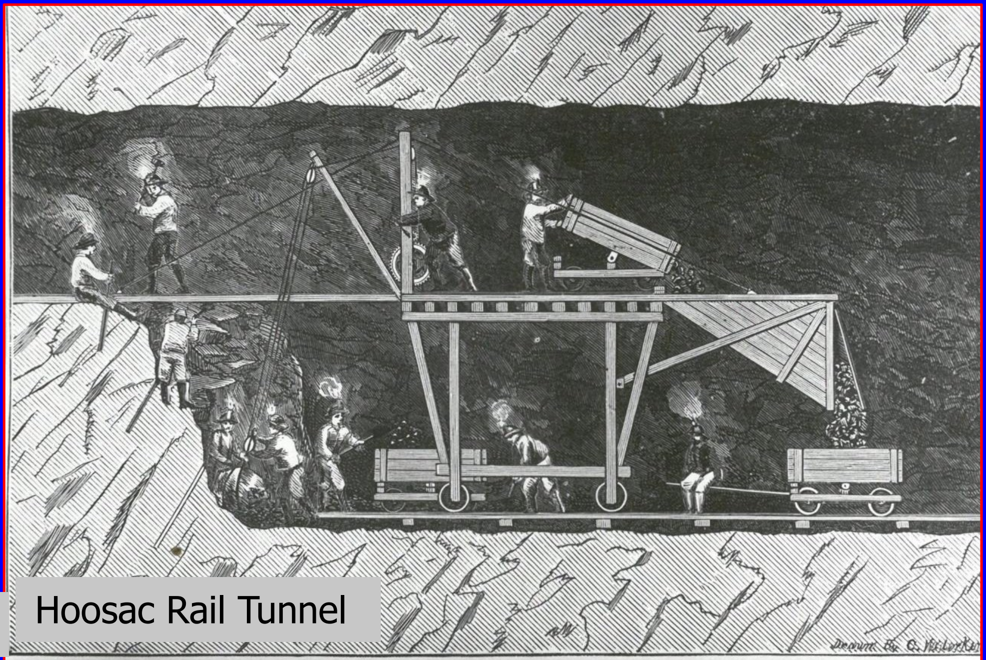
“Modern” tunneling began in late 1700s, mainly for canal navigation, with the use of black powder and various methods of timbering. Final linings, when used, included bricks, dressing stone and cement. In Great Britain, about 60 km of canal tunnels had been built by 1850. In France, a soft ground tunnel for a canal, with continuous brick arching, was completed in 1803.

In the early 1800s, consideration was given to tunneling under the Thames River in London, for communication across the river. The “shield”, patented by **Marc Isambard Brunel** in 1828, was used for the construction of the first Thames River Tunnel in the 1820s. The effort was so time consuming, expensive and riddled with problems, that no other shield tunneling was attempted until the late 1860s.

Starting in the 1850’s, tunnels began to be built for railroads, vastly increasing both tunnel size and the need for tunneling through difficult ground. Foremost among these railroad projects was the 13.7 km long **Mont Cenis (Frejus) Rail Tunnel** across the Alps (France to Italy, opened in 1871) and the **Hoosac Rail Tunnel** in Western Massachusetts (7.6 km, opened in 1875)



Brunel’s shield



Hoosac Rail Tunnel

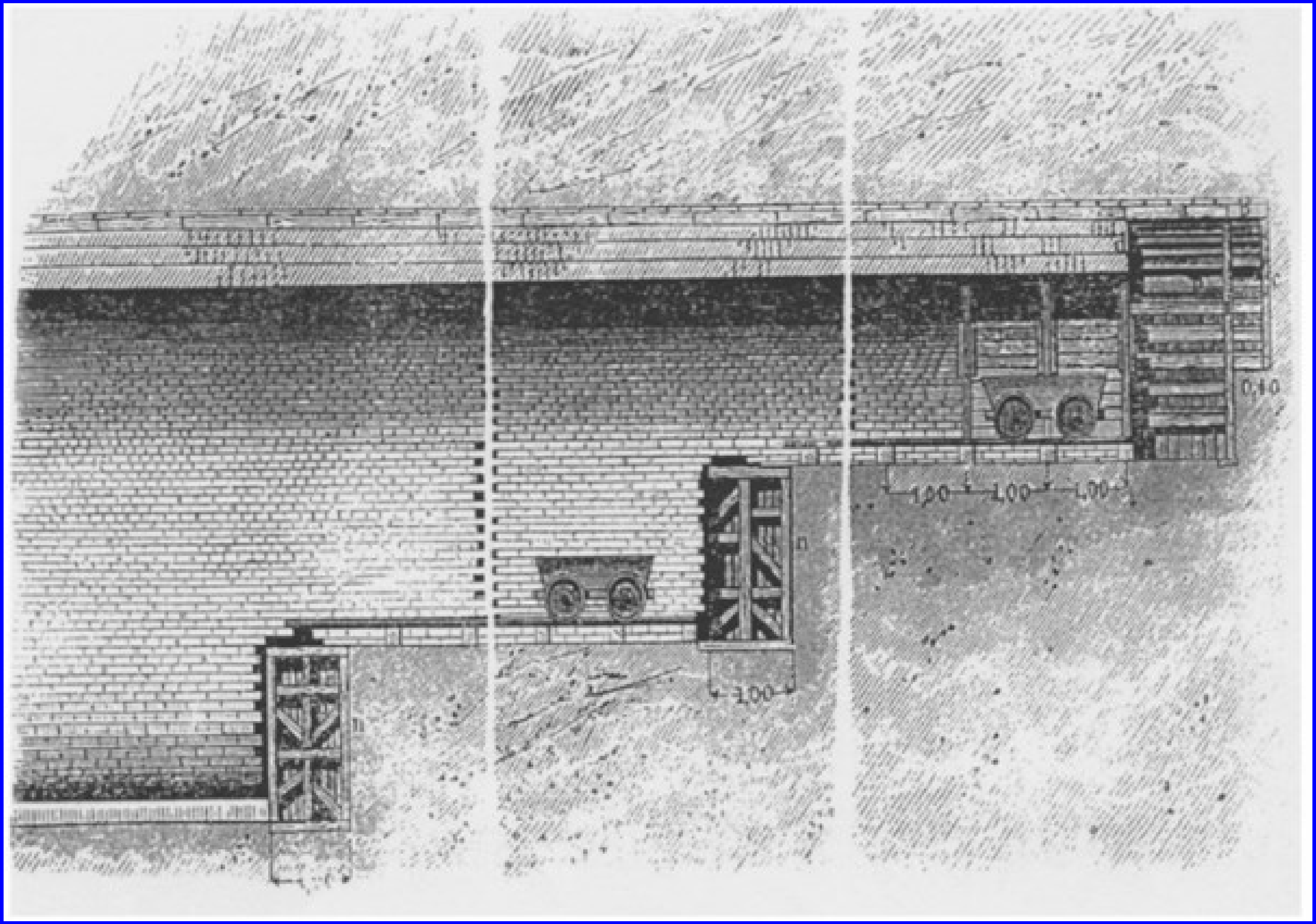
Mont Cenis (Frejus) Rail Tunnel across the Alps (France to Italy, 13.7 km, opened in 1871)

Portal from the Italian side

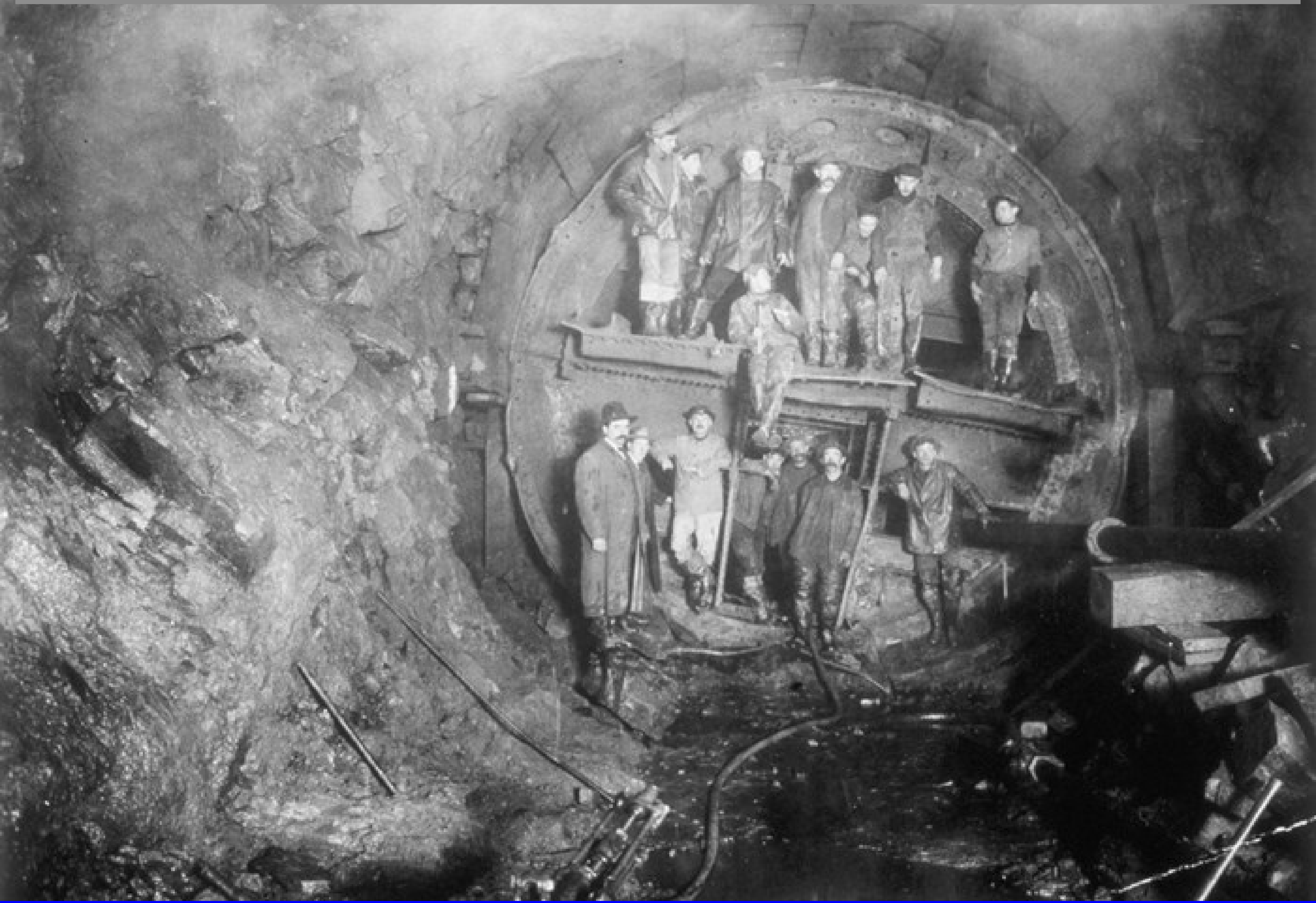
Original portal from the French side

Tunnel excavation methods

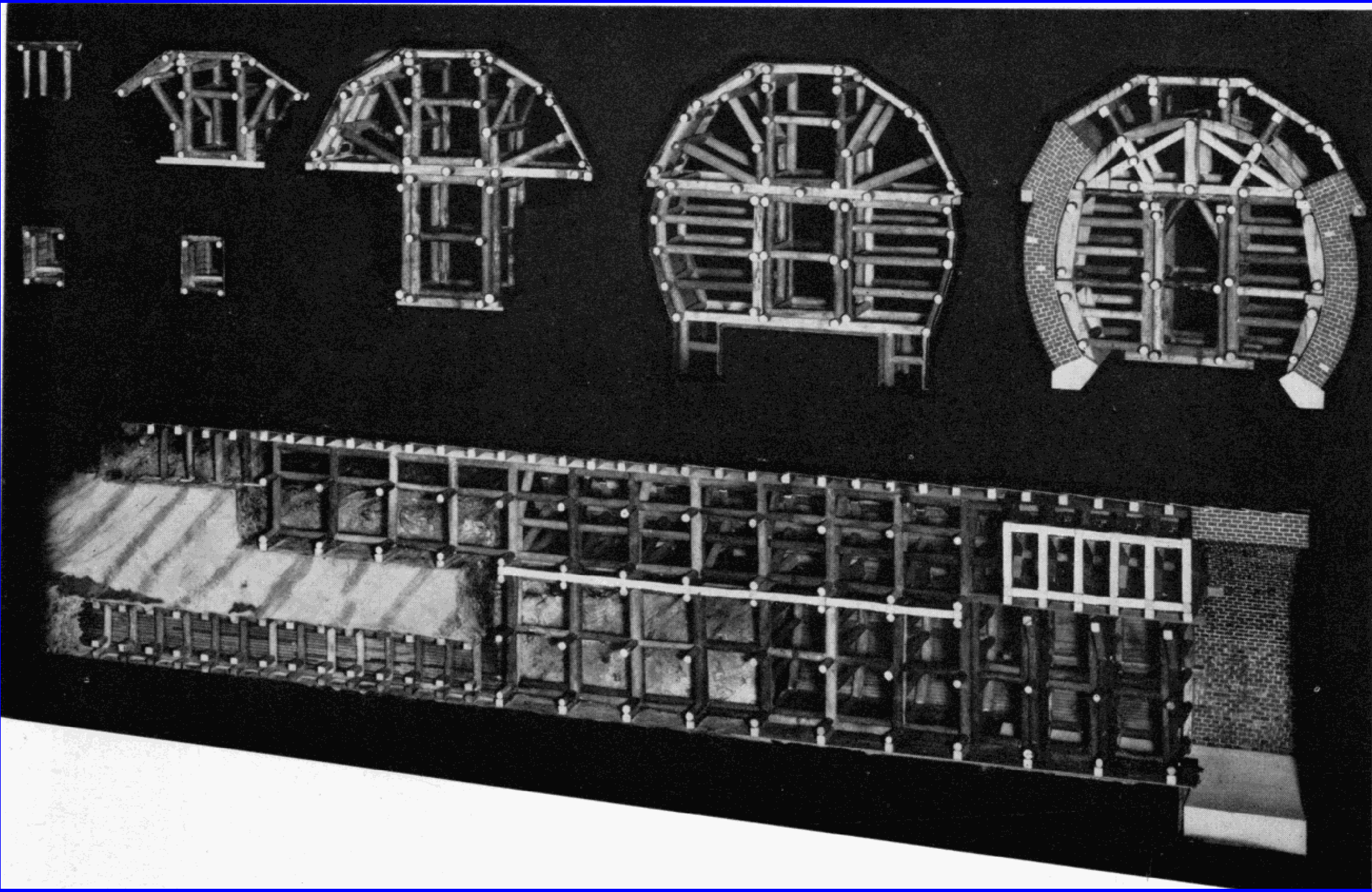
Old methods with brick support and multiple phases



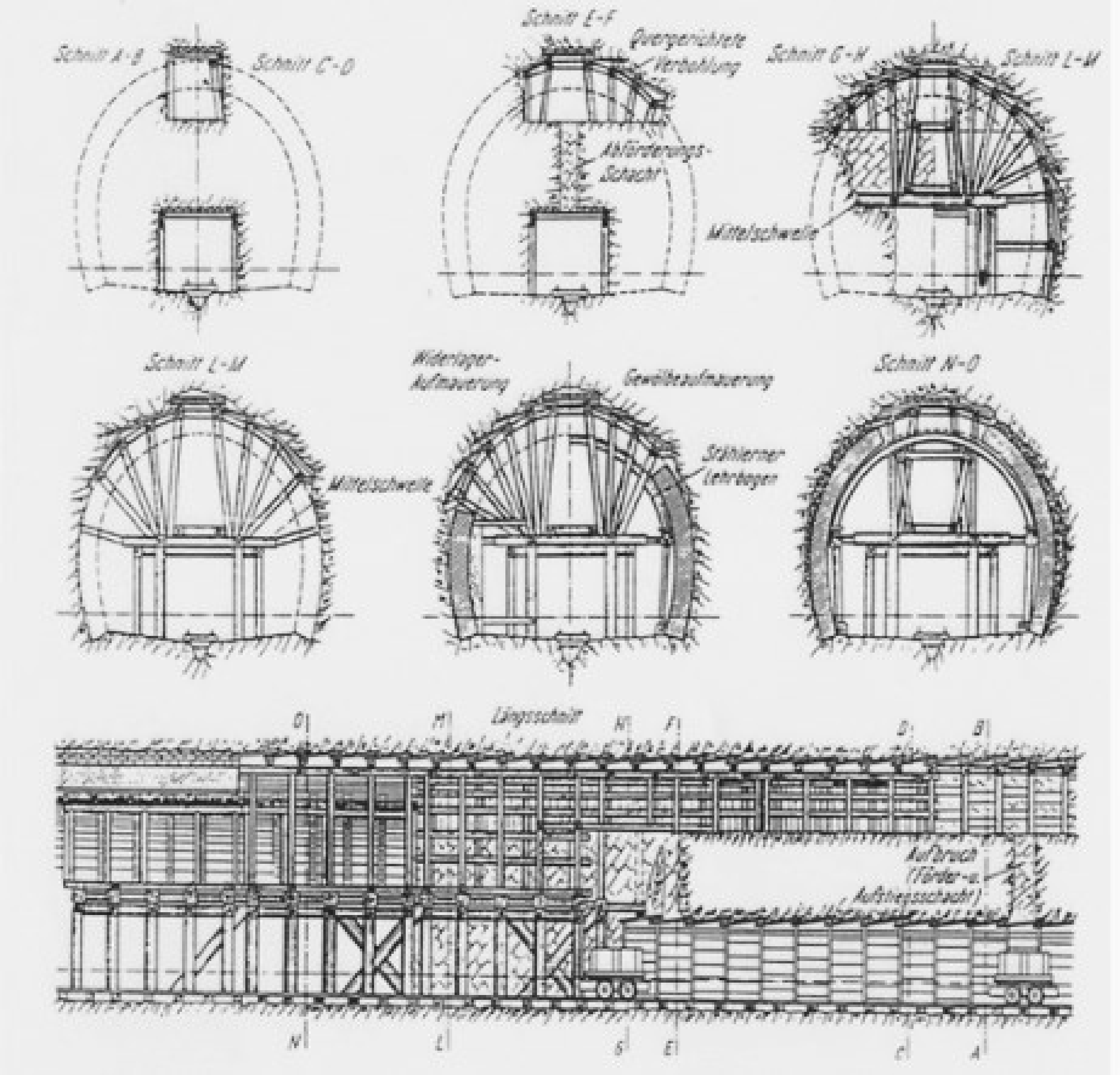
Shield-driven tunnels with cast-iron segments - Workers build early sections of the New York City subway in ca. 1900 (subway opened in 1904)



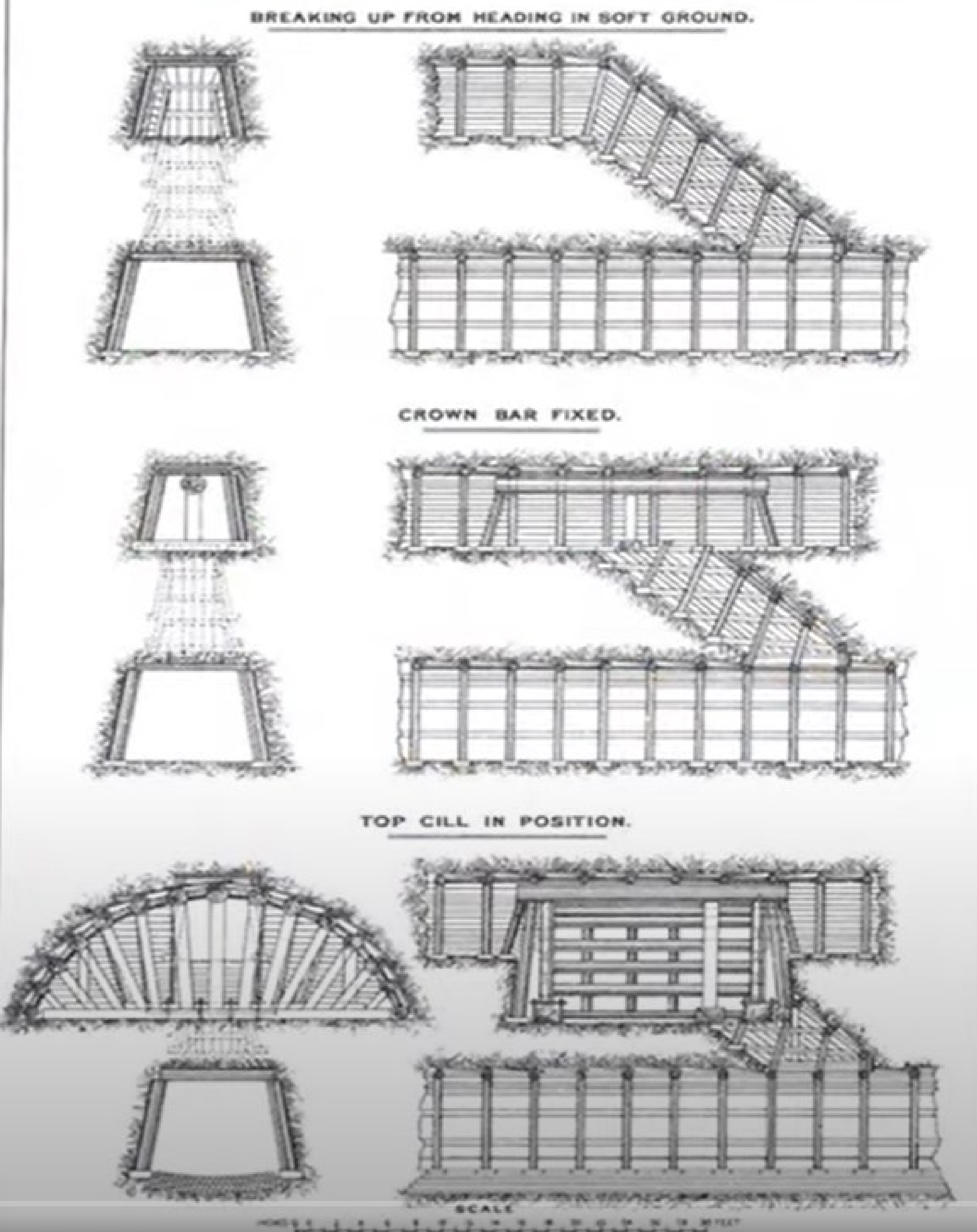
"Old" Austrian Tunneling Method



"Old" Austrian Tunneling Method



“Old” Austrian Tunneling Method



“Old” Austrian Tunneling Method

Old tunnel in a gold mine
(Olympiada, Greece)

Collapse of tunnels

ROCK TUNNELING

with

STEEL SUPPORTS

by

R. V. PROCTOR, M. E.

Vice President and General Manager
The Commercial Shearing & Stamping Co.

T. L. WHITE,

Registered Civil Engineer
Member, American Society of Mechanical Engineers

Chief Engineer of Design
The Commercial Shearing & Stamping Co.

with an

Terzaghi empirical loads (1946)

Introduction to Tunnel Geology

by

KARL TERZAGHI

Mem. Am. Soc. C. E.; Inst. C. E. (London)
Consulting Engineer

Youngstown, Ohio
1946

SECTION I

ROCK DEFECTS AND LOADS ON TUNNEL SUPPORTS

By

KARL TERZAGHI

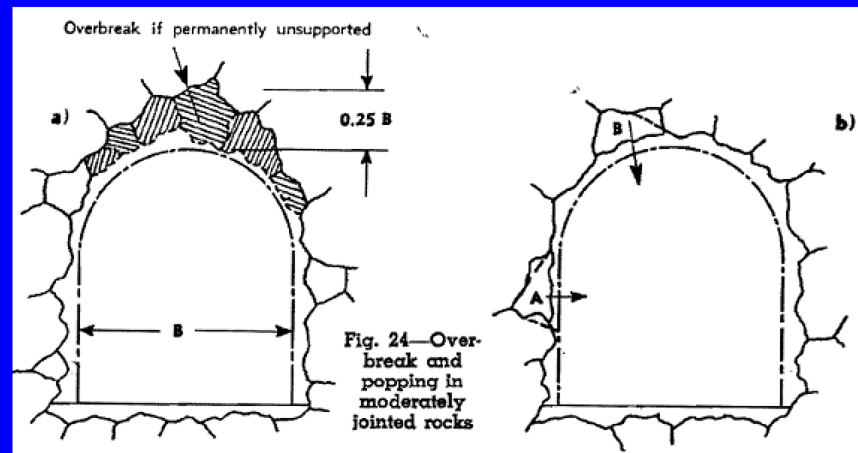


Fig. 24—Over-break and popping in moderately jointed rocks

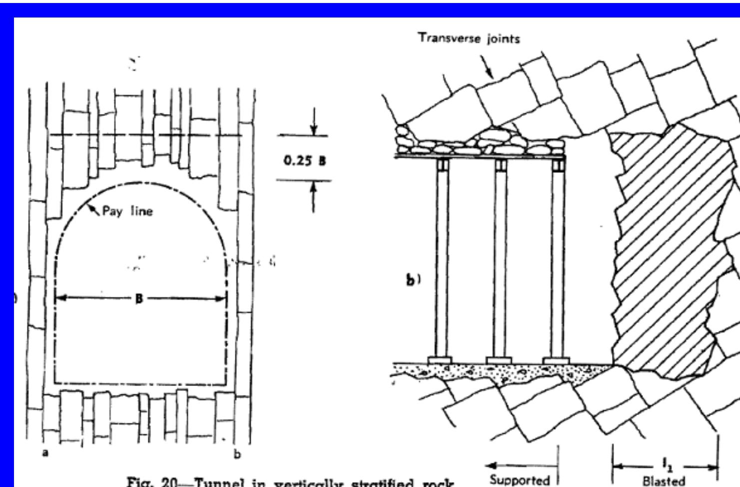


Fig. 20—Tunnel in vertically stratified rock

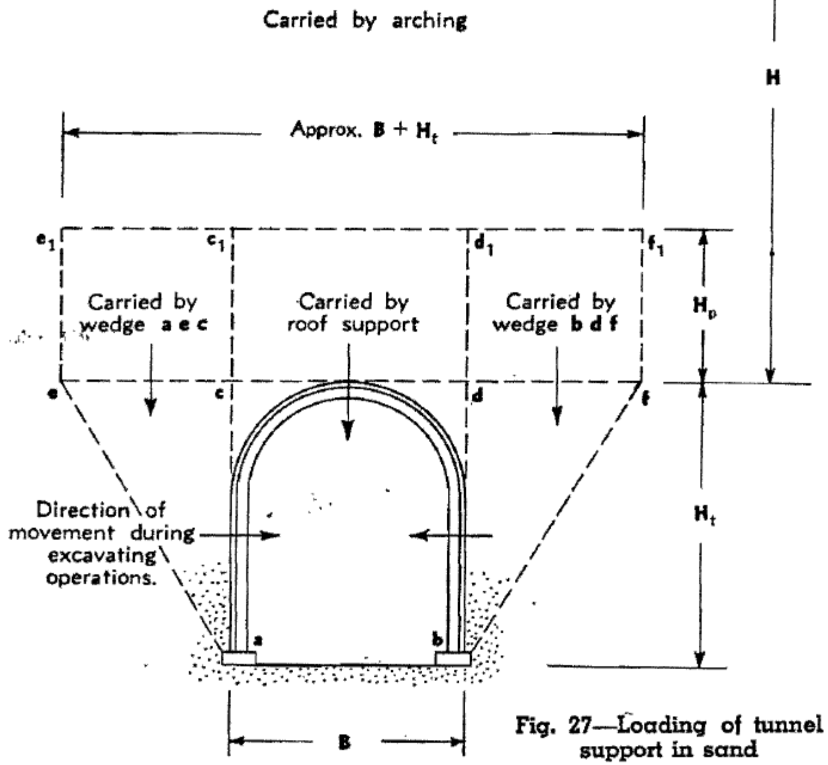


Fig. 27—Loading of tunnel support in sand

SECTION I

ROCK DEFECTS AND LOADS ON TUNNEL SUPPORTS

By

KARL TERZAGHI

TABLE 1

Rock Loads (in feet) in Blocky and Seamy Rock

	Initial value	*Ultimate value*
Moderately blocky rock	H_p = zero	$H_{p\ ult} = 0.25\ B$ to $0.35\ (B+H_t)$
Very blocky and shattered rock	H_p = zero to $0.60\ (B+H_t)$	$H_{p\ ult} = 0.35\ (B+H_t)$ to $1.10\ (B+H_t)$

TABLE 2

Comparison Between Rock Load (in feet) in Sand and in Blocky and Seamy Rock

Material		*Above water table*		*Below water table*[1]	
		$H_{p\ min}$	$H_{p\ max}$	$H_{p\ min}$	$H_{p\ max}$
Dense sand[2]	Initial	$0.27\ (B+H_t)$	$0.60\ (B+H_t)$	$0.54\ (B+H_t)$	$1.20\ (B+H_t)$
	Ultimate	$0.31\ (B+H_t)$	$0.69\ (B+H_t)$	$0.62\ (B+H_t)$	$1.38\ (B+H_t)$
Loose sand[2]	Initial	$0.47\ (B+H_t)$	$0.60\ (B+H_t)$	$0.94\ (B+H_t)$	$1.20\ (B+H_t)$
	Ultimate	$0.54\ (B+H_t)$	$0.69\ (B+H_t)$	$1.08\ (B+H_t)$	$1.38\ (B+H_t)$

Moderately blocky[3] $H_{p\ in} = 0$ increasing up to $H_{p\ ult} = 0.35\ (B+H_t)$

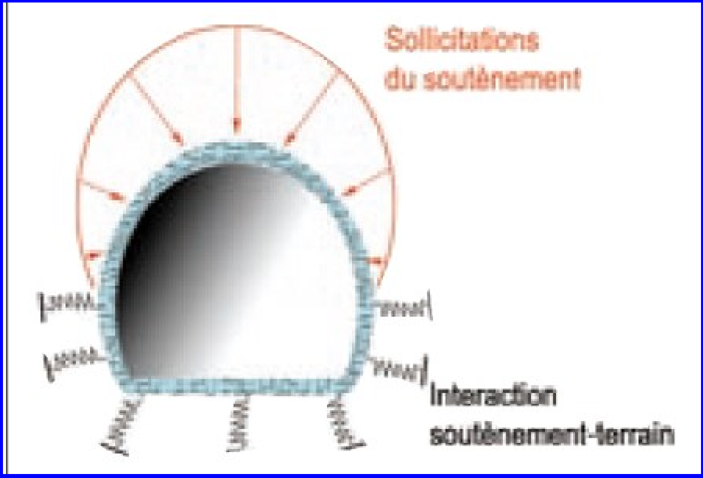
Very blocky and shattered $H_{p\ in} = .60\ (B+H_t)$ increasing up to $H_{p\ ult} = 1.10\ (B+H_t)$

1. Values are roughly equal to twice those for dry sand.
2. Values computed on basis of laboratory tests.
3. Values computed on the basis of the results of observations in railroad tunnels.

Conventional tunnel excavation

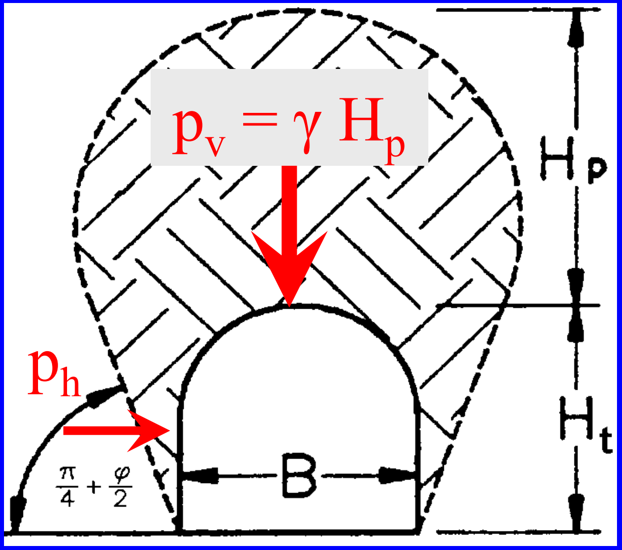
Older analysis methods of tunnel excavation and support design:

Analysis of the tunnel lining with known vertical and horizontal loads. In some cases, ground reaction springs are added to model the ground-structure interaction (added / reduced loads on the lining if it moves towards / away from the ground.



Terzaghi empirical loads (1946)

Tunnel: B (width), H_t (height) H_p = height of ground load



$p_h \approx 0.5\ p_v$

Rock Condition	RQD	Rock Load H_p
1. Hard and intact	95–100	Zero
2. Hard stratified or schistose	90–99	0–0.5 B
3. Massive, moderately jointed	85–95	0–0.25 B
4. Moderately blocky and seamy	75–85	0.25 B–0.20 $(B + H_t)$
5. Very blocky and seamy	30–75	(0.20–0.60) $(B + H_t)$
6. Completely crushed but chemically intact	3–30	(0.60–1.10) $(B + H_t)$
6a. Sand and gravel	0–3	(1.10–1.40) $(B + H_t)$
7. Squeezing rock, moderate depth	NA	(1.10–2.10) $(B + H_t)$
8. Squeezing rock, great depth	NA	(2.10–4.50) $(B + H_t)$
9. Swelling rock	NA	Up to 80m irrespective of value of $(B + H_t)$

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 1. Q-system (Norwegian method, 1974)

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

$\frac{RQD}{J_n}$ = Degree of jointing (or block size)

$\frac{J_r}{J_a}$ = Joint friction (inter-block shear strength)

$\frac{J_w}{SRF}$ = Active stress

RQD = Degree of jointing (Rock Quality Designation)

J_n = Joint set number

J_r = Joint roughness number

J_a = Joint alteration number

J_w = Joint water reduction factor

SRF = Stress Reduction Factor

2. $$\frac{\text{Span or height in m}}{\text{ESR}} = \text{Equivalent dimension}$$

Excavation Support Ratio (ESR)

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 1. Q-system (Norwegian method)

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

1	RQD (Rock Quality Designation)		RQD
A	Very poor	(> 27 joints per m^3)	0-25
B	Poor	(20-27 joints per m^3)	25-50
C	Fair	(13-19 joints per m^3)	50-75
D	Good	(8-12 joints per m^3)	75-90
E	Excellent	(0-7 joints per m^3)	90-100

Note: i) Where RQD is reported or measured as ≤ 10 (including 0) the value 10 is used to evaluate the Q-value

ii) RQD-intervals of 5, i.e. 100, 95, 90, etc., are sufficiently accurate

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 1. Q-system (Norwegian method)

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

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 joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random heavily jointed "sugar cube", etc	15
J	Crushed rock, earth like	20

Note: i) For tunnel intersections, use 3 x J_n

ii) For portals, use 2 x J_n

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 1. Q-system (Norwegian method)

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

3	Joint Roughness Number	J_r
a) Rock-wall contact, and *b) Rock-wall contact before 10 cm of shear movement*		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough, irregular, planar	1.5
F	Smooth, planar	1
G	Slickensided, planar	0.5
Note: i) Description refers to small scale features and intermediate scale features, in that order		
c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact when sheared	1
Note: ii) Add 1 if the mean spacing of the relevant joint set is greater than 3 m (dependent on the size of the underground opening) iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction		

4	Joint Alteration Number	Φ_r approx.	J_a
a) Rock-wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote.		0.75
B	Unaltered joint walls, surface staining only.	25-35°	1
C	Slightly altered joint walls. Non-softening mineral coatings; sandy particles, clay-free disintegrated rock, etc.	25-30°	2
D	Silty or sandy clay coatings, small clay fraction (non-softening).	20-25°	3
E	Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc gypsum, graphite, etc., and small quantities of swelling clays.	8-16°	4
b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc.	25-30°	4
G	Strongly over-consolidated, non-softening, clay mineral fillings (continuous, but <5 mm thickness).	16-24°	6
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5 mm thickness).	12-16°	8
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but <5 mm thickness). Value of J_a depends on percent of swelling clay-size particles.	6-12°	8-12
c) No rock-wall contact when sheared (thick mineral fillings)			
K	Zones or bands of disintegrated or crushed rock. Strongly over-consolidated.	16-24°	6
L	Zones or bands of clay, disintegrated or crushed rock. Medium or low over-consolidation or softening fillings.	12-16°	8
M	Zones or bands of clay, disintegrated or crushed rock. Swelling clay. J_a depends on percent of swelling clay-size particles.	6-12°	8-12
N	Thick continuous zones or bands of clay. Strongly over-consolidated.	12-16°	10
O	Thick, continuous zones or bands of clay. Medium to low over-consolidation.	12-16°	13
P	Thick, continuous zones or bands with clay. Swelling clay. J_a depends on percent of swelling clay-size particles.	6-12°	13-20

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design:

1. Q-system (Norwegian method)

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

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 1. Q-system (Norwegian method)

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

5	Joint Water Reduction Factor	J_w
A	Dry excavations or minor inflow (humid or a few drips)	1.0
B	Medium inflow, occasional outwash of joint fillings (many drips/"rain")	0.66
C	Jet inflow or high pressure in competent rock with unfilled joints	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	0.33
E	Exceptionally high inflow or water pressure decaying with time. Causes outwash of material and perhaps cave in	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay. Causes outwash of material and perhaps cave in	0.1-0.05

Note: i) Factors C to F are crude estimates. Increase J_w if the rock is drained or grouting is carried out

ii) Special problems caused by ice formation are not considered

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 1. Q-system (Norwegian method)

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

6	**Stress Reduction Factor**			**SRF**
a)	*Weak zones intersecting the underground opening, which may cause loosening of rock mass*			
A	Multiple occurrences of weak zones within a short section containing clay or chemically disintegrated, very loose surrounding rock (any depth), or long sections with incompetent (weak) rock (any depth). For squeezing, see 6L and 6M			10
B	Multiple shear zones within a short section in competent clay-free rock with loose surrounding rock (any depth)			7.5
C	Single weak zones with or without clay or chemical disintegrated rock (depth ≤ 50m)			5
D	Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)			5
E	Single weak zones with or without clay or chemical disintegrated rock (depth > 50m)			2.5
Note:	i) Reduce these values of SRF by 25-50% if the weak zones only influence but do not intersect the underground opening			
b)	*Competent, mainly massive rock, stress problems*	σ_c / σ_1	σ_θ / σ_c	**SRF**
F	Low stress, near surface, open joints	>200	<0.01	2.5
G	Medium stress, favourable stress condition	200-10	0.01-0.3	1
H	High stress, very tight structure. Usually favourable to stability. May also be unfavourable to stability dependent on the orientation of stresses compared to jointing/weakness planes*	10-5	0.3-0.4	0.5-2 2-5*
J	Moderate spalling and/or slabbing after > 1 hour in massive rock	5-3	0.5-0.65	5-50
K	Spalling or rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
L	Heavy rock burst and immediate dynamic deformation in massive rock	<2	>1	200-400
Note:	ii) For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.75\,\sigma_c$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c to $0.5\,\sigma_c$, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ = maximum tangential stress (estimated from elastic theory) iii) When the depth of the crown below the surface is less than the span; suggest SRF increase from 2.5 to 5 for such cases (see F)			

c)	*Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure*	σ_θ / σ_c	**SRF**
M	Mild squeezing rock pressure	1-5	5-10
N	Heavy squeezing rock pressure	>5	10-20
Note:	iv) Determination of squeezing rock conditions must be made according to relevant literature (i.e. Singh et al., 1992 and Bhasin and Grimstad, 1996)		
d)	*Swelling rock: chemical swelling activity depending on the presence of water*		**SRF**
O	Mild swelling rock pressure		5-10
P	Heavy swelling rock pressure		10-15

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 1. Q-system (Norwegian method)

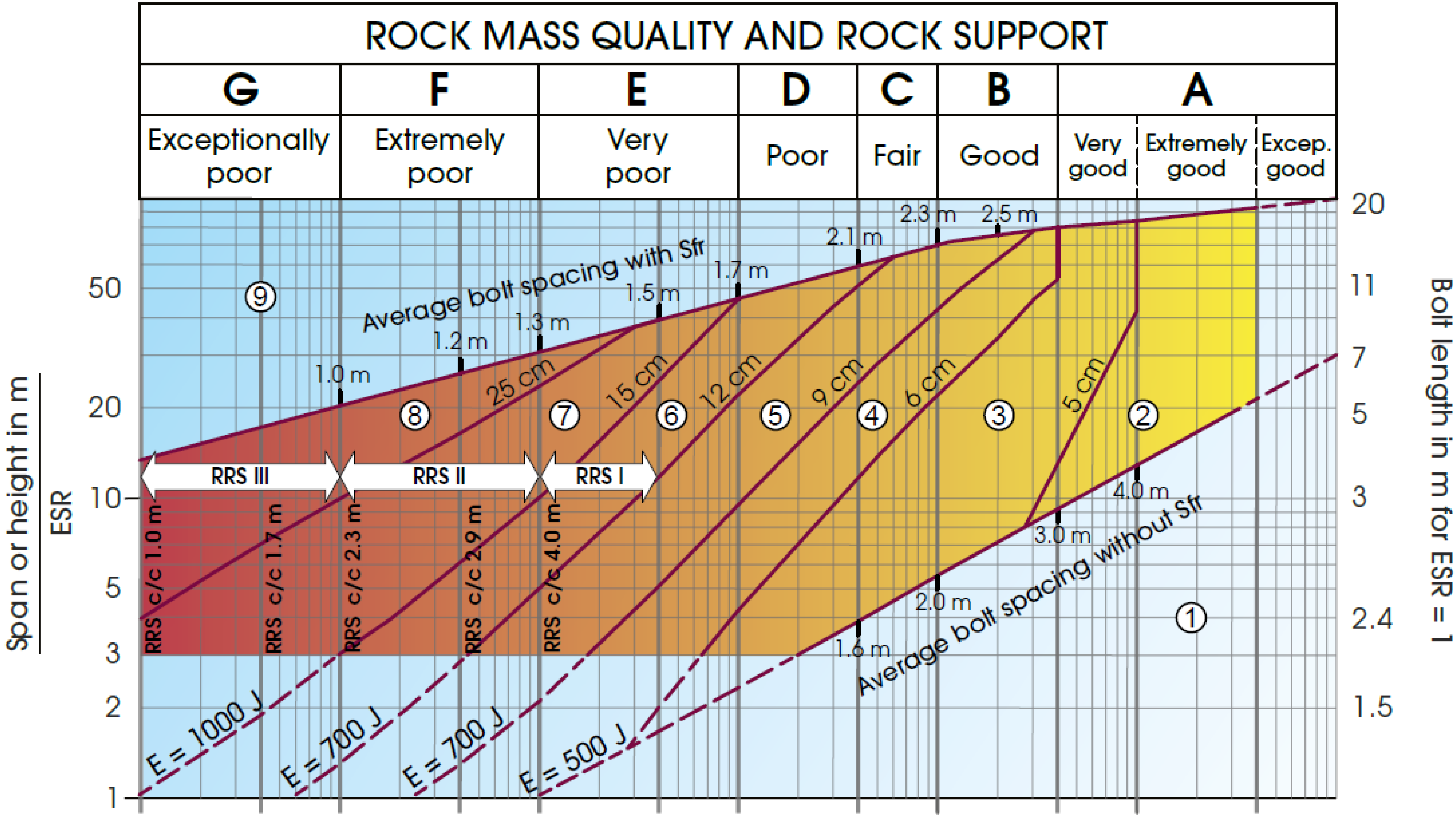
$$\frac{\text{Span or height in m}}{\text{ESR}} = \text{Equivalent dimension}$$

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

Excavation Support Ratio (ESR)

7	Type of excavation	ESR
A	Temporary mine openings, etc.	*ca.* 3-5
B	Vertical shafts*: i) circular sections ii) rectangular/square section * Dependant of purpose. May be lower than given values.	*ca.* 2.5 *ca.* 2.0
C	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), water supply tunnels, pilot tunnels, drifts and headings for large openings.	1.6
D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels, etc.	1.3
E	Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
F	Underground nuclear power stations, railways stations, sports and public facilitates, factories, etc.	0.8
G	Very important caverns and underground openings with a long lifetime, ≈ 100 years, or without access for maintenance.	0.5

Conventional tunnel excavation (NATM)
Empirical methods of tunnel support design: 1. Q-system (Norwegian method)



Sfr = Shotcrete (fibre reinforced)

RRS = Rib Reinforced Shotcrete

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

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 1. Q-system (Norwegian method)

Support categories

① Unsupported or spot bolting
② Spot bolting, **SB**
③ Systematic bolting, fibre reinforced sprayed concrete, 5-6 cm, **B+Sfr**
④ Fibre reinforced sprayed concrete and bolting, 6-9 cm, **Sfr (E500)+B**
⑤ Fibre reinforced sprayed concrete and bolting, 9-12 cm, **Sfr (E700)+B**
⑥ Fibre reinforced sprayed concrete and bolting, 12-15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E700)+RRS I +B**
⑦ Fibre reinforced sprayed concrete >15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E1000)+RRS II+B**
⑧ Cast concrete lining, **CCA** or **Sfr (E1000)+RRS III+B**
⑨ Special evaluation

Bolts spacing is mainly based on Ø20 mm

E = Energy absorbtion in fibre reinforced sprayed concrete

ESR = Excavation Support Ratio

Areas with dashed lines have no empirical data

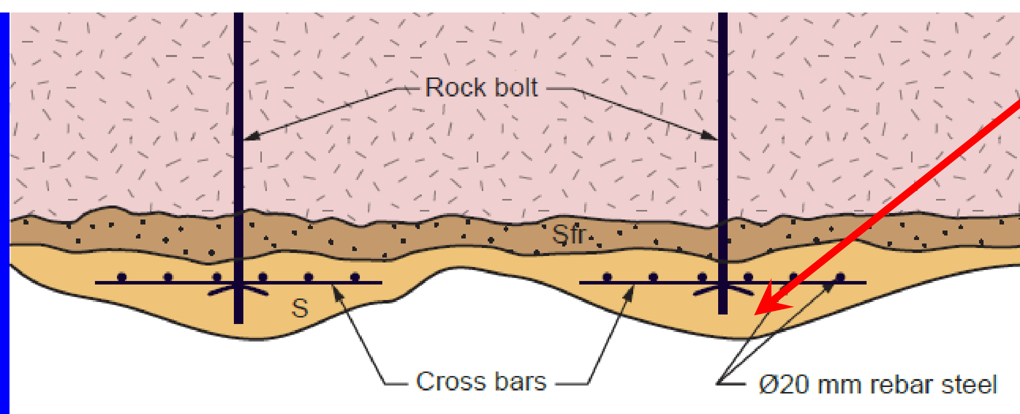
RRS - spacing related to Q-value

I
Si30/6 Ø16 - Ø20 (span 10m)
D40/6+2 Ø16-20 (span 20m)

II
Si35/6 Ø16-20 (span 5m)
D45/6+2 Ø16-20 (span 10m)
D55/6+4 Ø20 (span 20m)

III
D40/6+4 Ø16-20 (span 5 m)
D55/6+4 Ø20 (span 10 m)
Special evaluation (span 20 m)

Si30/6 = Single layer of 6 rebars, 30 cm thickness of sprayed concrete
D = Double layer of rebars
Ø16 = Rebar diameter is 16 mm
c/c = RSS spacing, centre - centre



RRS (Rib Reinforced Shotcrete

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 2. Rock Mass Rating (RMR) system (1976)

$$RMR = R1 + R2 + R3 + R4 + R5 + R6$$

TABLE 4.1 The Rock Mass Rating System (Geomechanics Classification of Rock Masses)[a]

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point-load strength index (MPa)	>10	4 – 10	2 – 4	1 – 2	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength (MPa)	>250	100 – 250	50 – 100	25 – 50	5 – 25	1 – 5	<1
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD (%)		90 – 100	75 – 90	50 – 75	25 – 50	<25		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6 – 2 m	200 – 600 mm	60 – 200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered wall	Slickensided surfaces or Gouge < 5 mm thick or Separation 1 – 5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (L/min)	None	<10	10 – 25	25 – 125	>125		
			or	or	or	or	or		
		Ratio $\frac{\text{Joint water pressure}}{\text{Major principal stress}}$	0	<0.1	0.1 – 0.2	0.2 – 0.5	>0.5		
			or	or	or	or	or		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 2. Rock Mass Rating (RMR) system

$$RMR = R1 + R2 + R3 + R4 + R5 + R6$$

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS

Strike and Dip Orientations of Discontinuities		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels and mines	0	−2	−5	−10	−12
	Foundations	0	−2	−7	−15	−25
	Slopes	0	−5	−25	−50	−60

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.

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	<20
Class no.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 2. Rock Mass Rating (RMR) system

$$RMR = R1 + R2 + R3 + R4 + R5 + R6$$

		Support		
Rock Mass Class	Excavation	Rock Bolts (20-mm Dia, Fully Grouted)	Shotcrete	Steel Sets
Very good rock I RMR:81–100	Full face 3-m advance	Generally, no support required except for occasional spot bolting		
Good rock II RMR:61–80	Full face 1.0–1.5-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
Fair rock III 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
Poor rock IV 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 wall with wire mesh	100–150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
Very poor rock V 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 fore-poling if required. Close invert

Shape: horseshoe; width: 10 m; vertical stress: <25 MPa; construction: drilling and blasting.

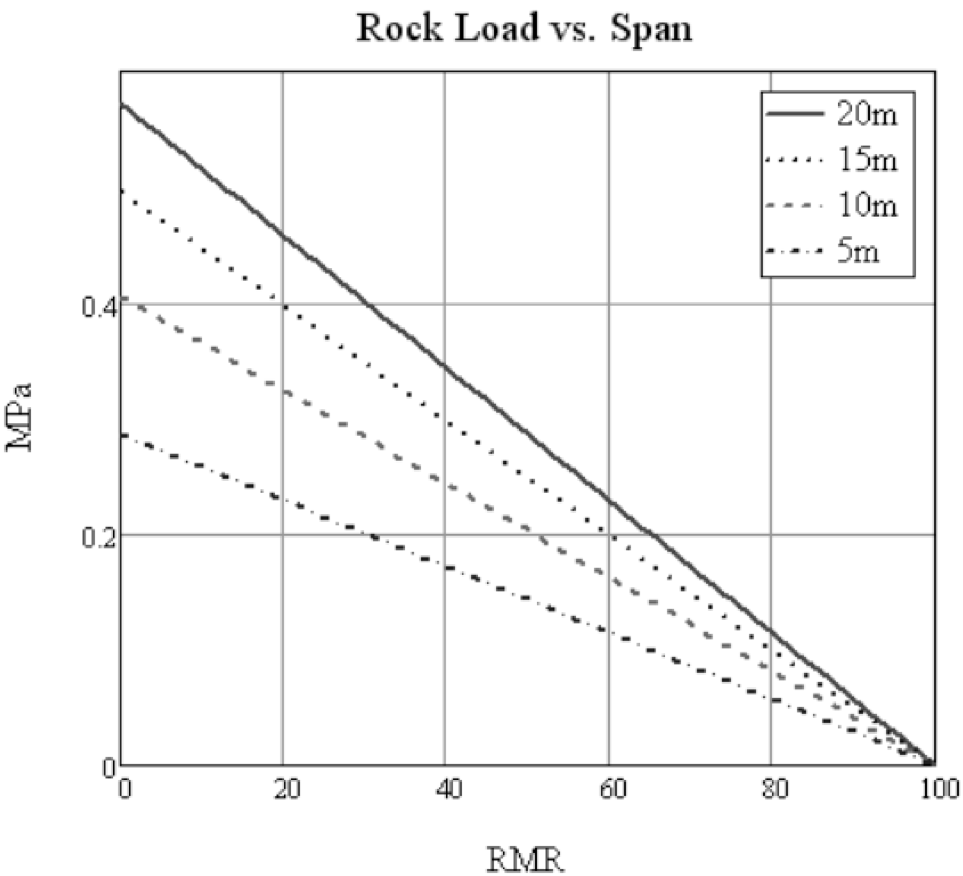
Conventional tunnel excavation (NATM)

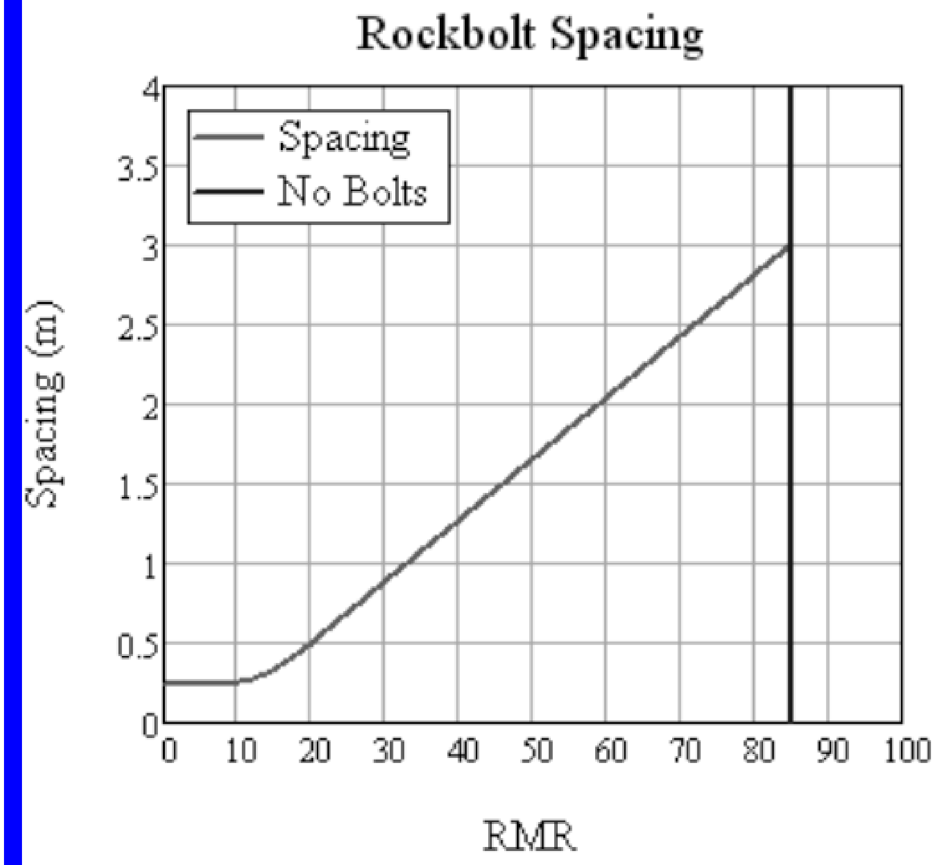
Empirical methods of tunnel support design: 2. Rock Mass Rating (RMR) system

$$RMR = R1 + R2 + R3 + R4 + R5 + R6$$

Design Rock Load:
$$P_r = \frac{100 - RMR}{100} . 10m . \left(\frac{Span}{10m}\right)^{\frac{1}{2}} . \rho_r . \gamma_r$$

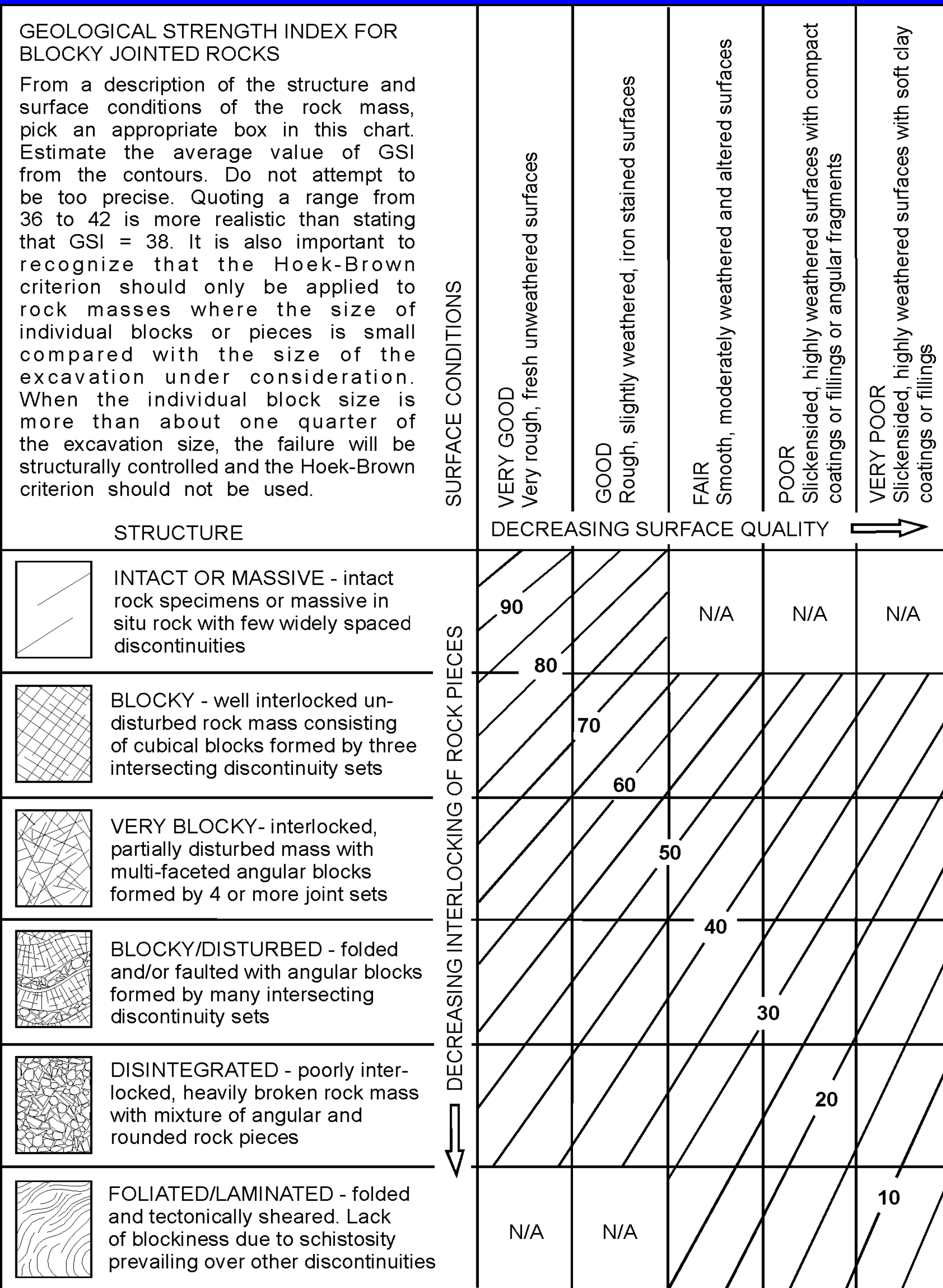
γ_r is a partial factor and ρ_r is rock density. For $\gamma_r = 1.5$ and $\rho_r = 27 kN/m^3$





Conventional tunnel excavation (NATM)

Empirical methods of tunnel support design: 3. Geological Strength Index (GSI)



Hoek (1996)

GSI is estimated by a direct method (a double entry Table), and thus involves less uncertainty than Q or RMR (which are the product or sum of many factors, and thus uncertainties add).

The GSI is not used to obtain support measures directly, but to <u>calculate mechanical parameters</u> (strength and stiffness) by empirical rules. These parameters are then used in numerical methods of analysis of tunnel excavation to obtain the required support measures.

E.g. modulus calculation:

$$E = \sqrt{\frac{\sigma_{ci}}{100}}\, alog\left(\frac{GSI-10}{40}\right)$$

Strength calculation:

$$\sigma_{cm} = \left(\frac{\sigma_{ci}}{52.63}\right)\exp\left(\frac{GSI}{20}\right)$$

“New” Austrian Tunneling Method (NATM)

Beginning with the late 1950’s, the use of **rockbolts** and **shotcrete** for support, revolutionized tunnelling in difficult ground.
This technique first gained attention in the work of Rabcewicz, Müller & Pacher between 1957 and 1964 in Austria, who named it “Shotcrete Method”. In 1964, Rabcewicz named it “New Austrian Tunnelling Method” (NATM)

WATER POWER November 1964

The New Austrian Tunnelling Method

After describing the influence of rock-pressure effects on tunnel linings, the author underlines the inadequacy of conventional tunnel driving and lining methods in poor ground and explains the effectiveness and reliability of a new method consisting of a thin sprayed concrete lining, closed at the earliest possible moment by an invert to a complete ring—called an "auxiliary arch"—the deformation of which is measured as a function of time until equilibrium is obtained. Ways are shown to determine the magnitude of active forces, which leads to dimensioning of linings on an empirical basis*. Further articles describe successful applications of the method

By Prof. Dr.techn. L. v. RABCEWICZ

PART ONE

The New Austrian Tunnelling Method

In this second article the author describes a number of actual tunnels, in various countries, in the construction of which the new Austrian method has been applied successfully

By Prof. Dr. techn. L. v. RABCEWICZ

PART TWO

The New Austrian Tunnelling Method

In this final article the author stresses the value of rock deformation measurements in determining the thickness of the lining. A test tunnel has been constructed in Austria to investigate this subject. A failed tunnel construction which was rescued by applying the new Austrian technique is described

By Prof. Dr. techn. L. v. RABCEWICZ

PART THREE

IN the conventional tunnelling practice of the past, masonry in dressed stone or brick was regarded as the most suitable lining material in unstable rock. Concrete was rejected because possible deformation during the settling and hardening process was supposed to cause irreparable damage. The space between masonry lining and rock face was dry packed. Timber lagging, which was subject to decay when left in place, generally could not be removed, particularly from the roof, because of the danger of loosening and rockfalls.

The situation was further aggravated by a very unfavourable time factor. Merely to bring to full section a 9m-long section of a double-track railway tunnel by the old Austrian tunnelling method, after the bottom and top headings had been driven, took about four weeks, and another month was needed to complete the masonry of the section. The amount of timber used in more difficult cases was so enormous that one third and sometimes even more of the excavated space was filled by solid timber.

Modern Tunnelling Methods

Finally, during the last few decades, rockbolting and shotcrete* were introduced in tunnelling practice. To judge from the results obtained up to now the introduction of these methods of support and surface protection can be considered as a most important event, especially in the field of soft-rock and earth tunnelling†.

The advantages of these methods can best be shown by comparing the rock mechanics of tunnels lined by the new and by older methods. Whereas all the older methods of temporary support without exception are bound to cause loosening and voids by yielding of the different parts of the supporting structure, a thin layer of shotcrete together with a suitable system of rockbolting applied to the rock face immediately after blasting entirely prevents loosening and reduces decompression to a certain degree, transforming the surrounding rock into a self-supporting arch.

Conventional tunnel excavation (NATM)

Face excavation

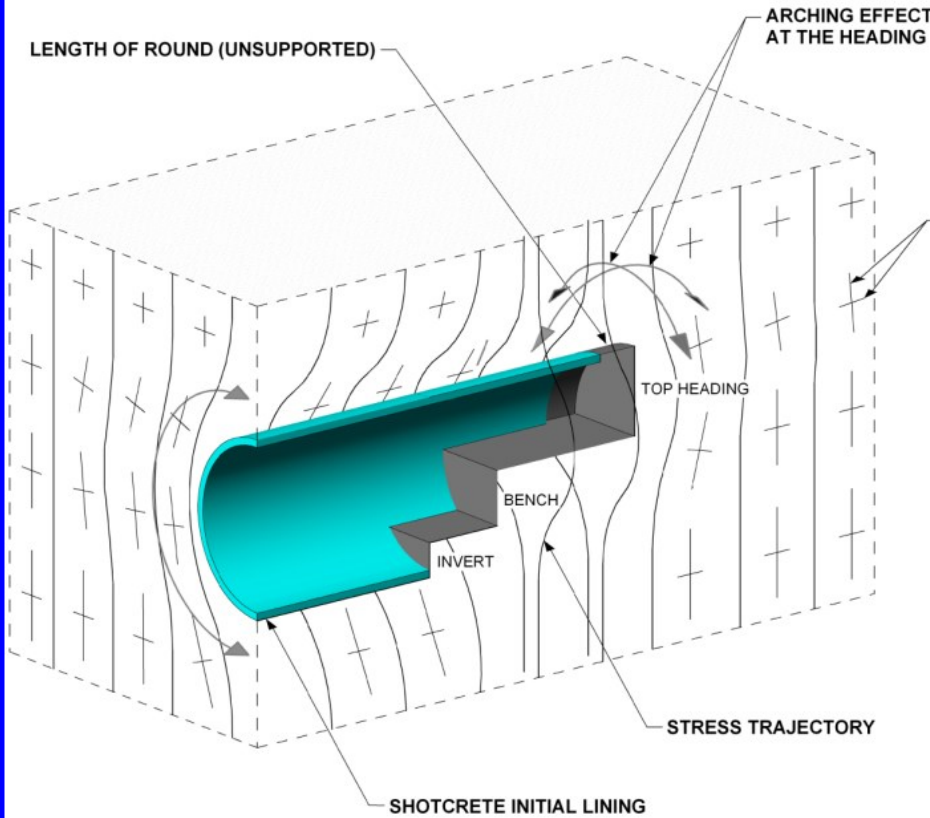

Face support with nails

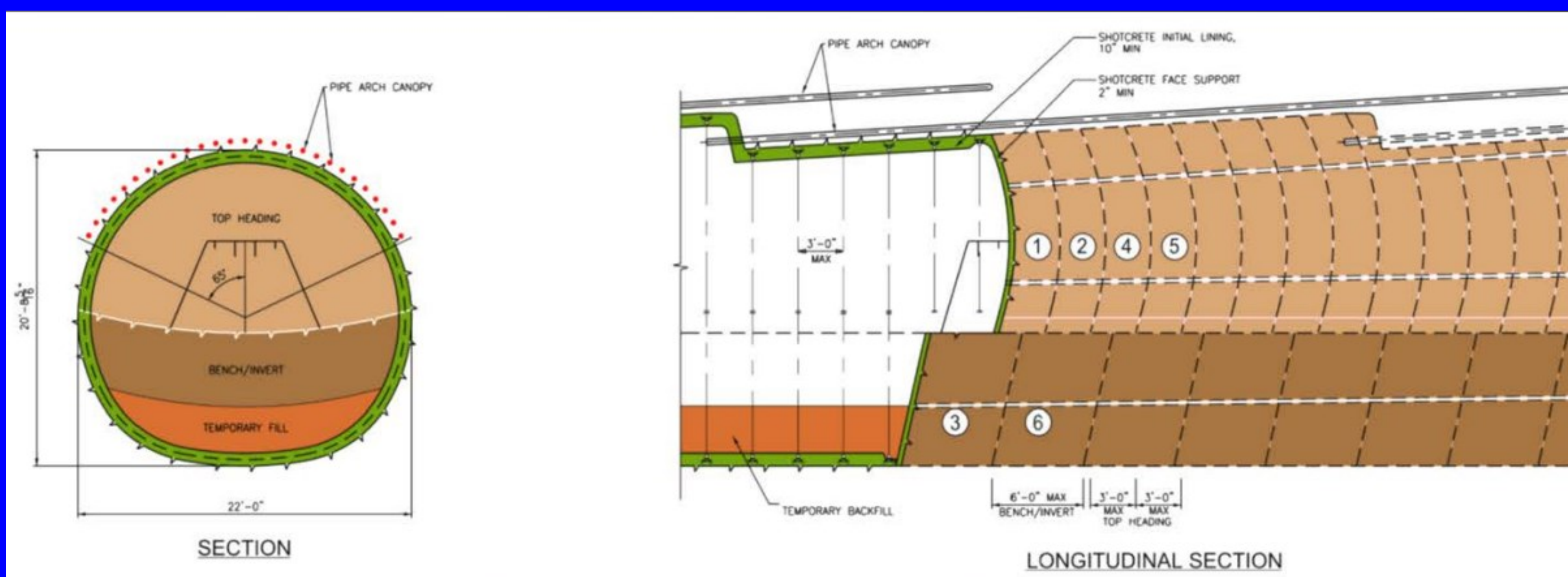

Face charging with explosives


Face support with forepoling

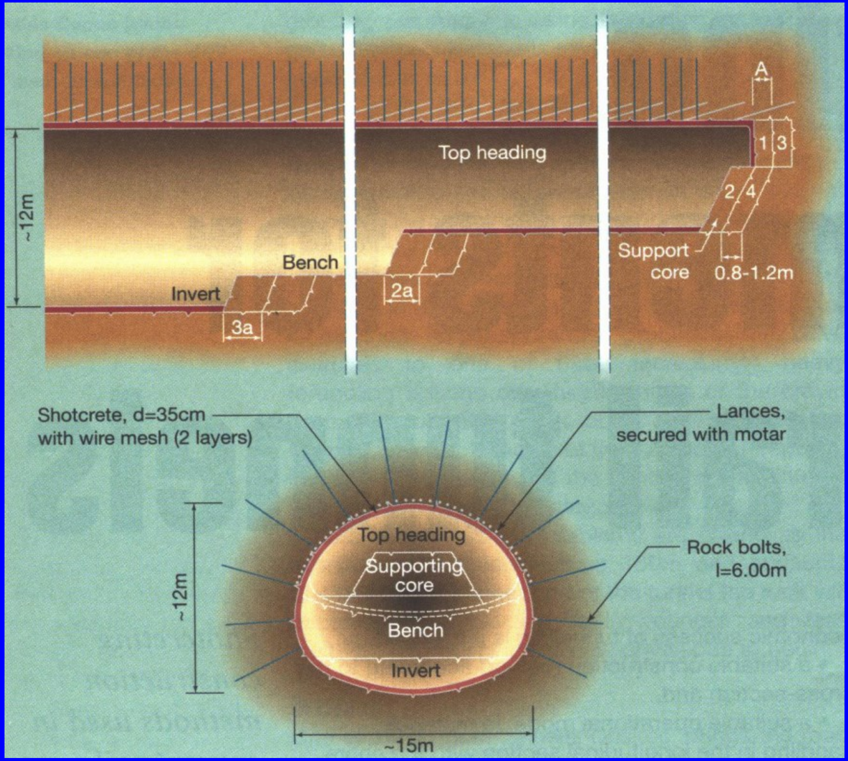
Conventional tunnel excavation (NATM)

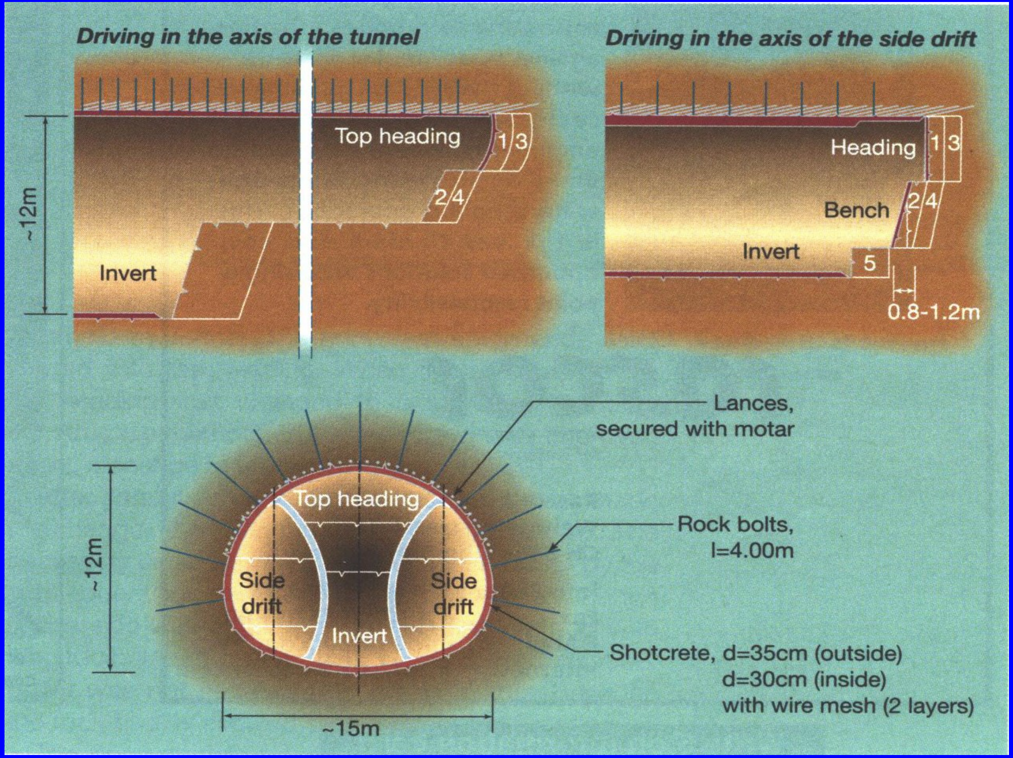
Multi-phase excavation and support, to reduce the size of each excavation (and better control face and wall stability and deformations)



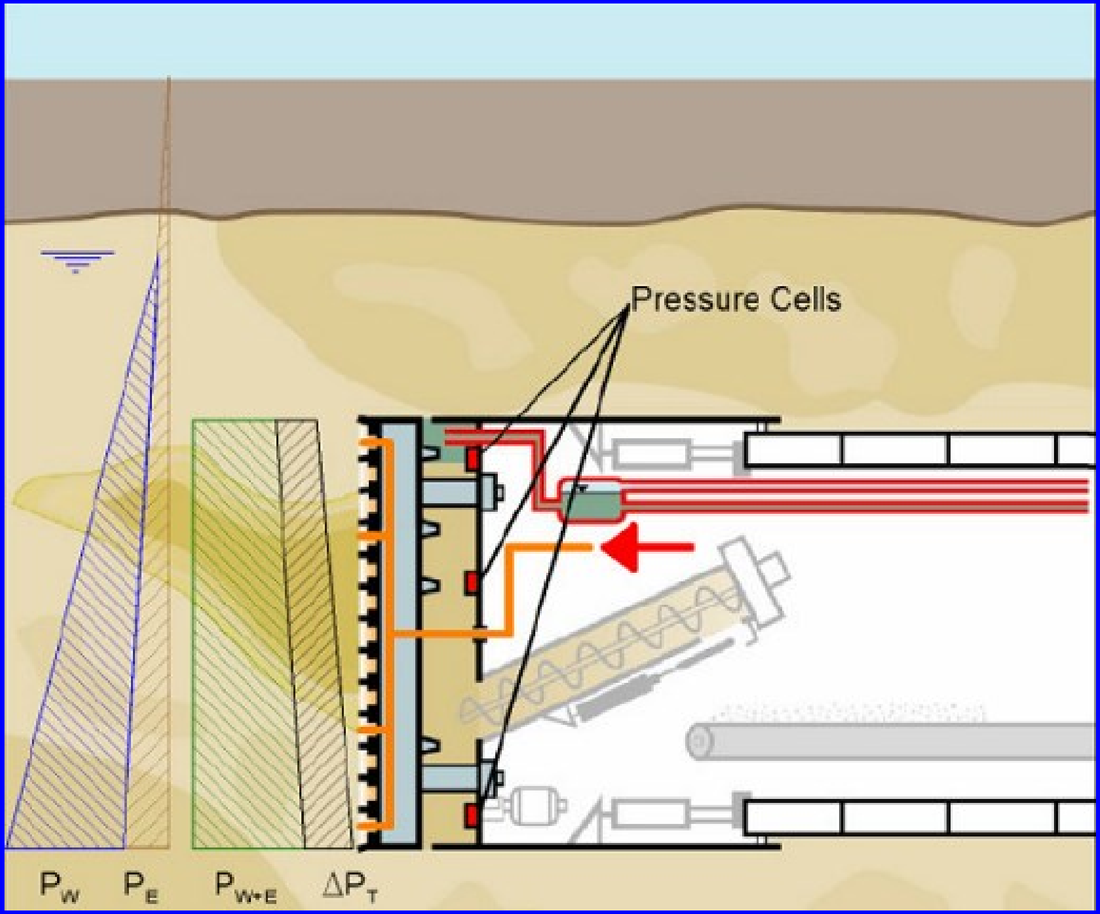


Conventional tunnel excavation (NATM)

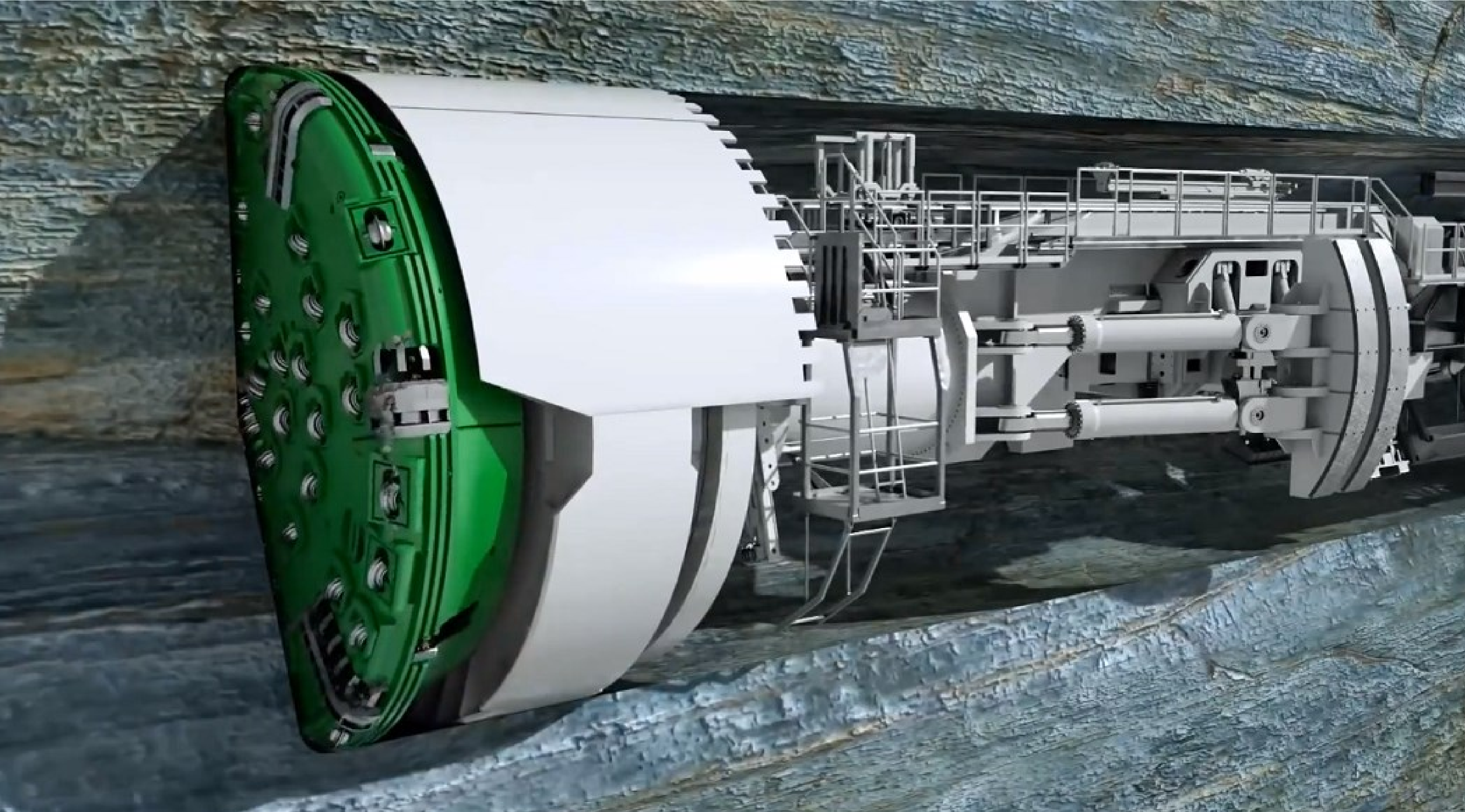


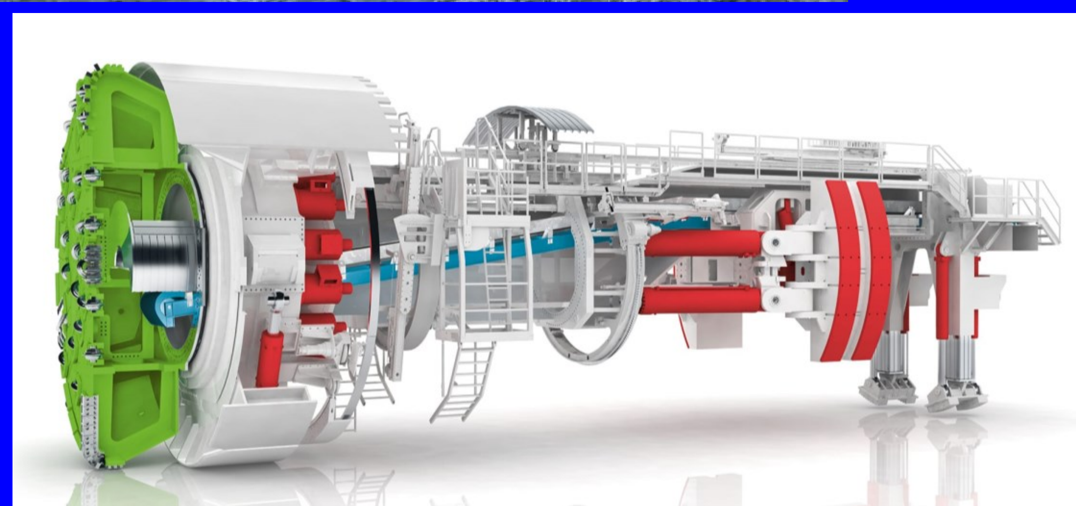


Mechanised tunnel excavation (TBM)



- Has better control (minimization) of face extrusion and radial wall convergence. Thus. has advantage over conventional excavation (NATM) in urban tunnels
- Faster production rate (meters per day). Has advantage over NATM in long tunnels
- An “industrialised” method with advantages (faster excavation rate) and disadvantages (less flexible in changing ground). Advantages prevail in long tunnels (> 2500m), since the high cost of the TBM machine is spread over a long length.





Rock (Gripper, open) TBM

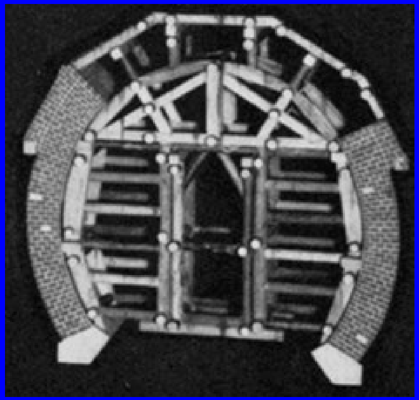
Soft ground TBMs
(single shield, double shield, EPB, slurry, etc)

Mechanised tunnel excavation (TBM)
Segmental tunnel lining

Mechanised tunnel excavation (TBM)

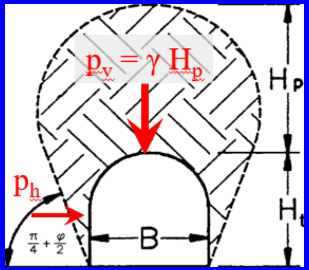
Partial Face TBM (Roadheaders)

History of tunnel design methods

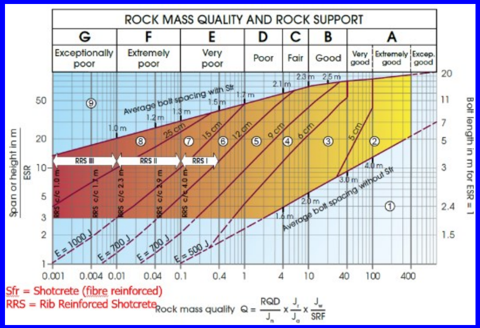


Purely empirical methods of tunnelling (1850's – 1946)

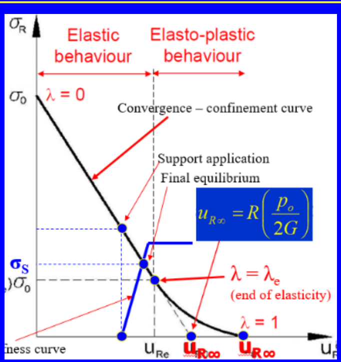
Terzaghi loads (1946)



Empirical design methods
Q-system (1974), RMR (1976)

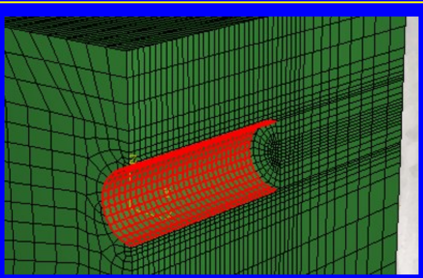


Convergence-Confinement method
(Pacher, 1964 or Panet, 1974)



GSI (1994)

3D Numerical analyses (2010's)



2D Numerical analyses (1990's)