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)

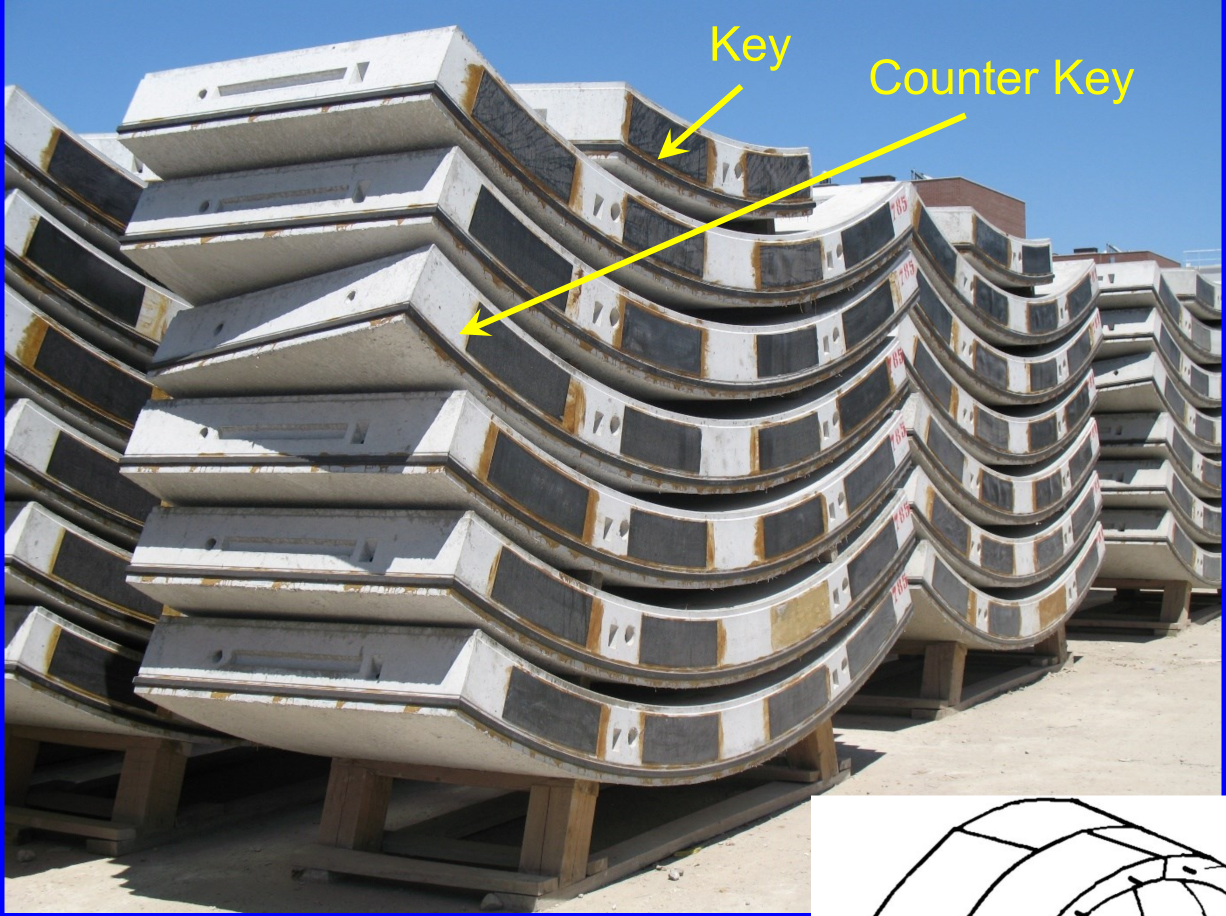
Instructor: Michael Kavvadas, Emer. Professor NTUA

LECTURE 6: Final Lining of tunnels

21.05.2024

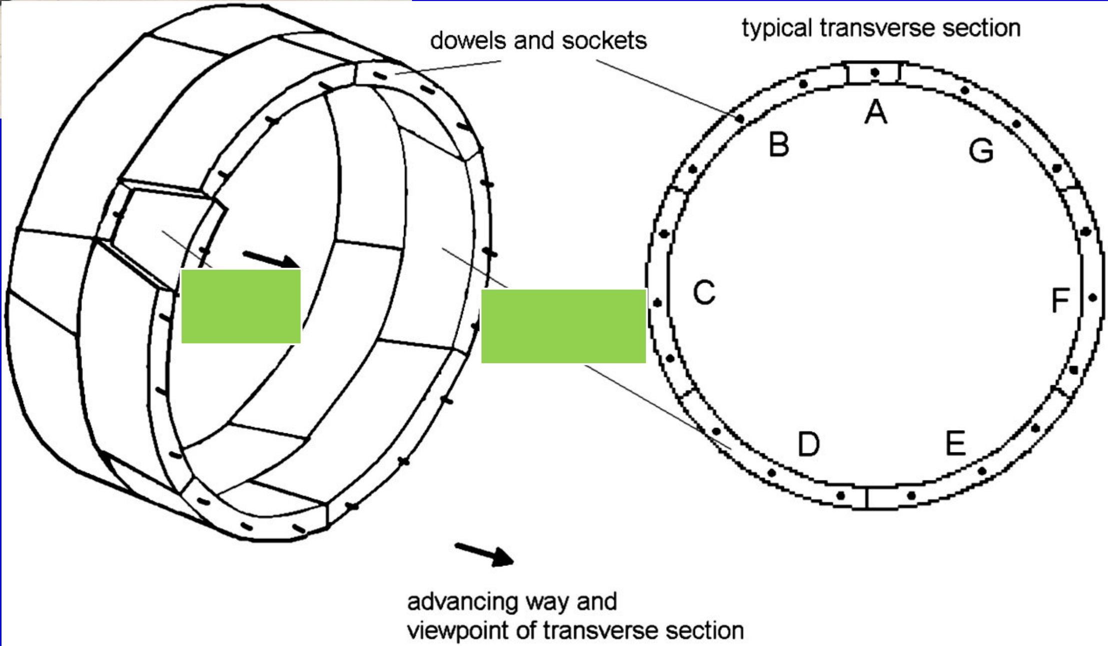
Final lining with precast segments - TBM excavated tunnel

Precast segmental lining for a TBM tunnel



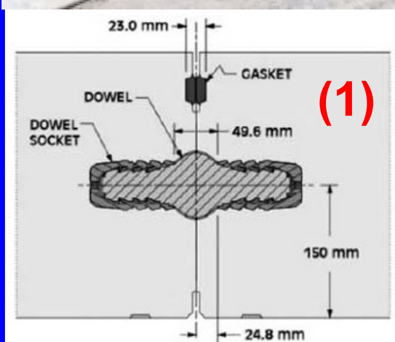
Short key segment, locks the ring

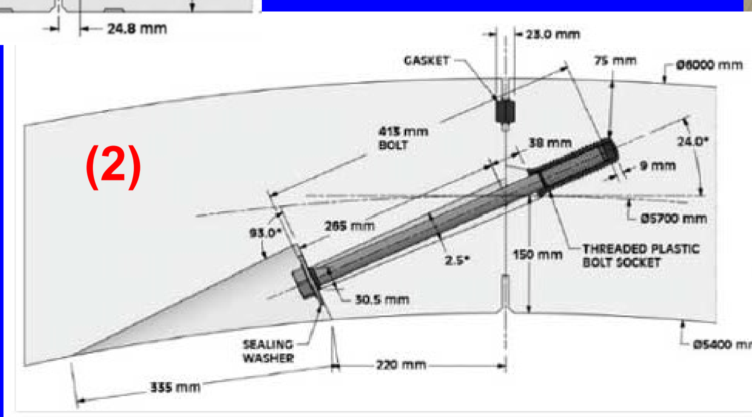
Counter key jointed segments are used to accommodate turns in the tunnel alignment



Precast segmental lining for a TBM tunnel

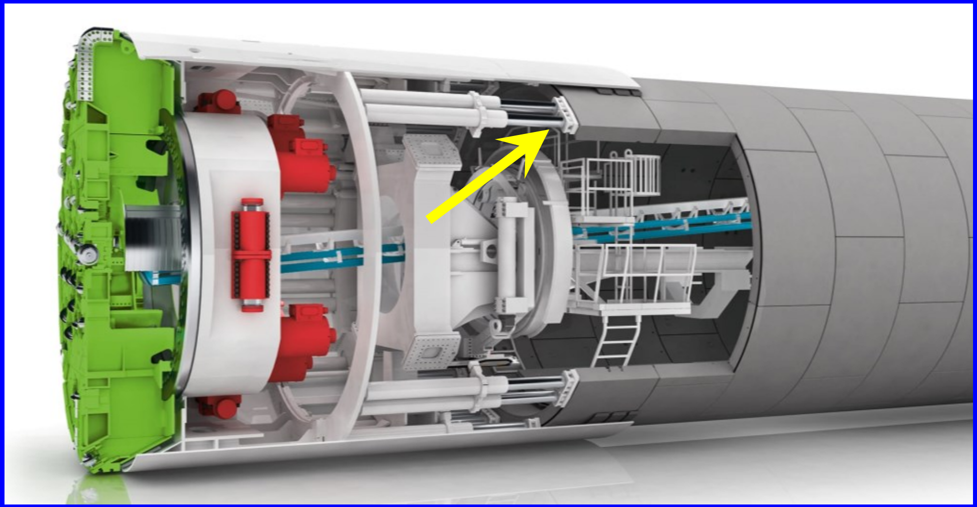
Waterproofing gaskets and connection holes (with dowels)





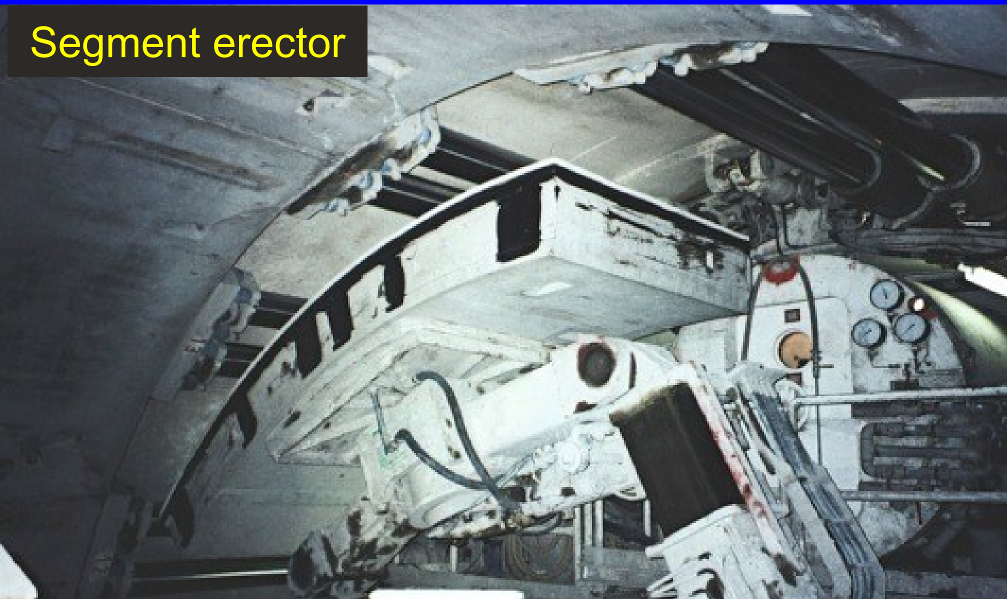
Transportation of prefabricated segments


Erection of the segmental lining in a TBM tunnel

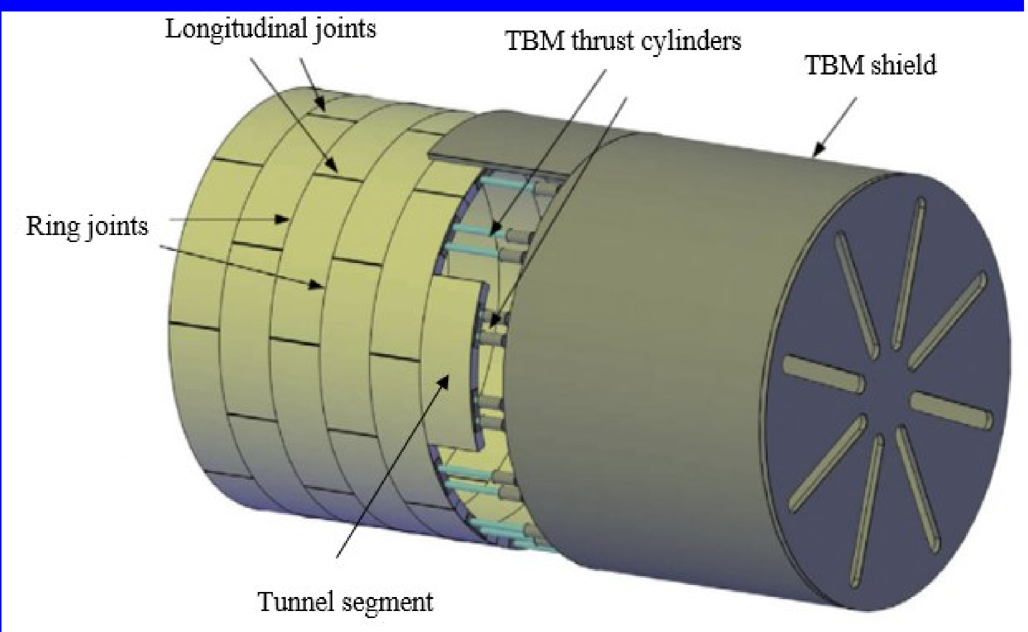



Segment erector

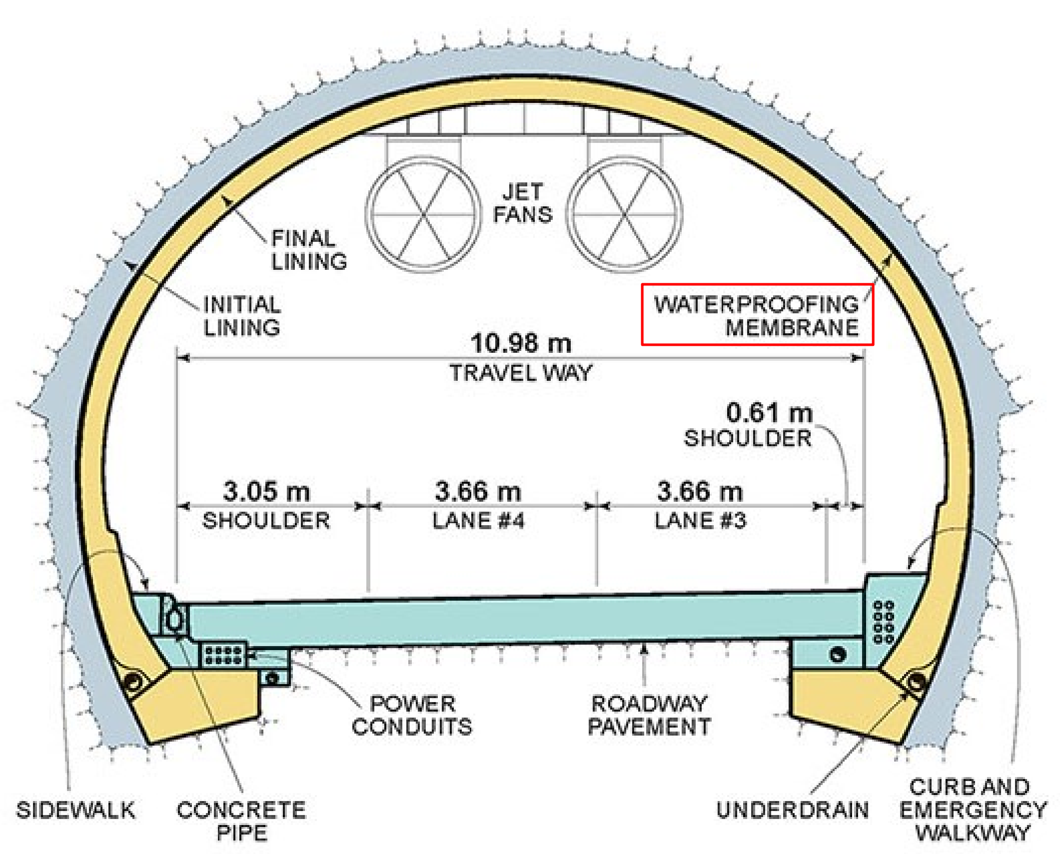
Erection of the segmental lining in a TBM tunnel



TBM thrust on segmental lining

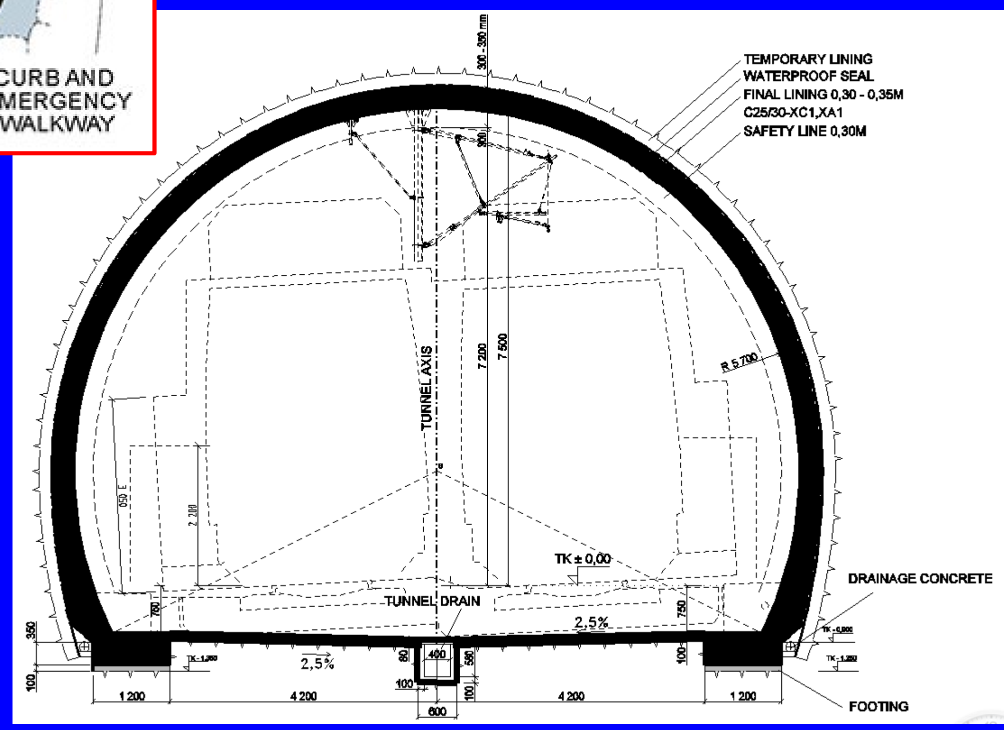


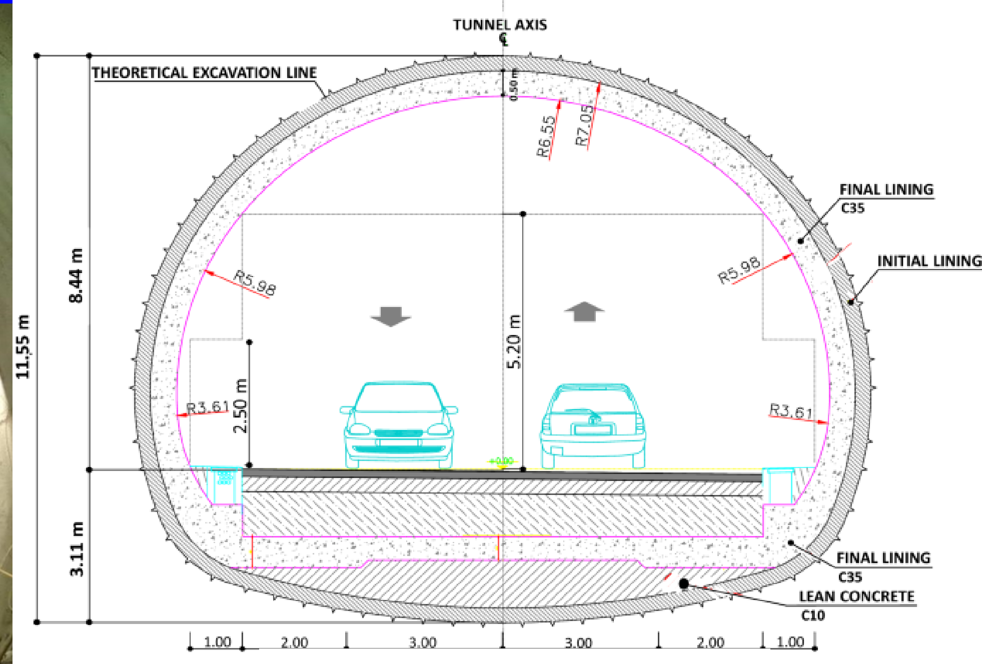
Cast in-situ final lining of tunnels with open invert (in good ground)



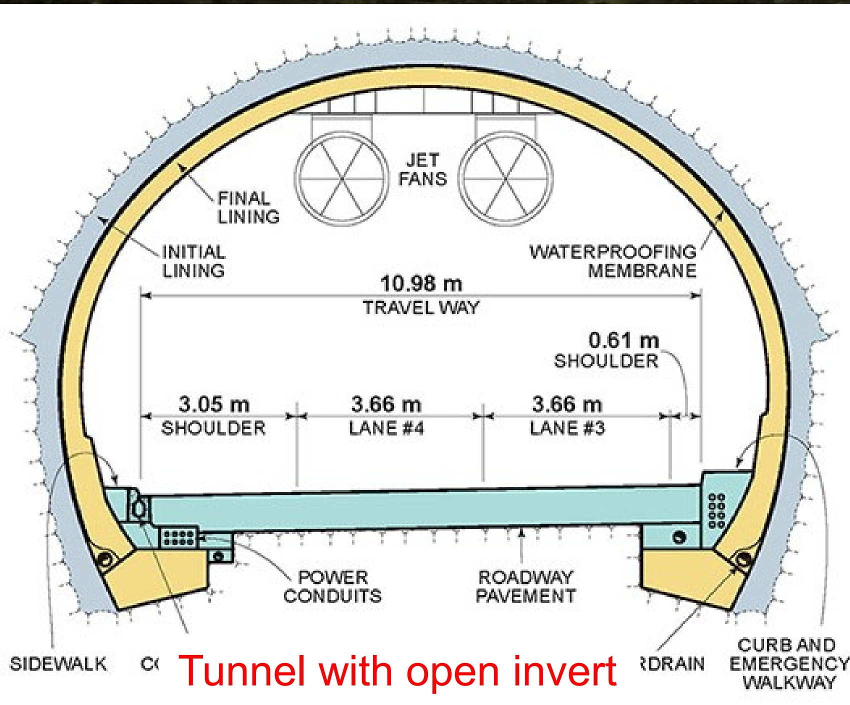
Road tunnel

Railway tunnel (double track)





Tunnel with closed invert



Tunnel with open invert

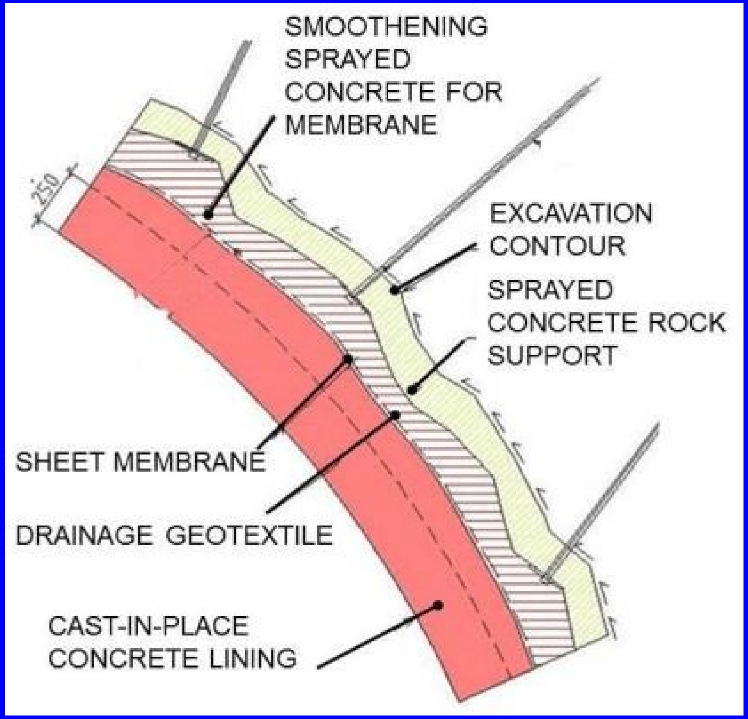
Cast-in-place final lining of tunnels

Primary support

Protection and drainage geotextile

Water-proofing synthetic membrane

Cast-in-place RC final lining



Final lining of tunnels – Construction of steel reinforcement at the invert

Final lining of tunnels – Construction of steel reinforcement at the invert

Final lining of tunnels – Invert after concreting

Final lining of tunnels – Placement of the water-proofing membrane

Final lining of tunnels – Placement of the water-proofing membrane

Final lining of tunnels – Placement of the water-proofing membrane

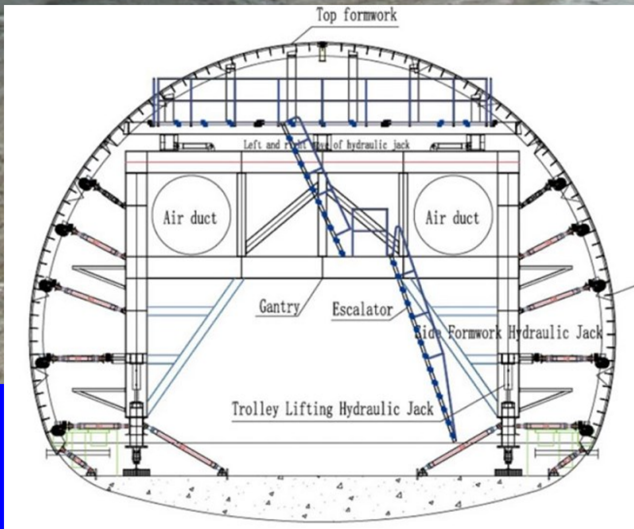
Final lining of tunnels – Placement of the water-proofing membrane and the steel reinforcement

Final lining of tunnels – Erection of steel reinforcement for the portal

Final lining of tunnels – Erection of steel reinforcement

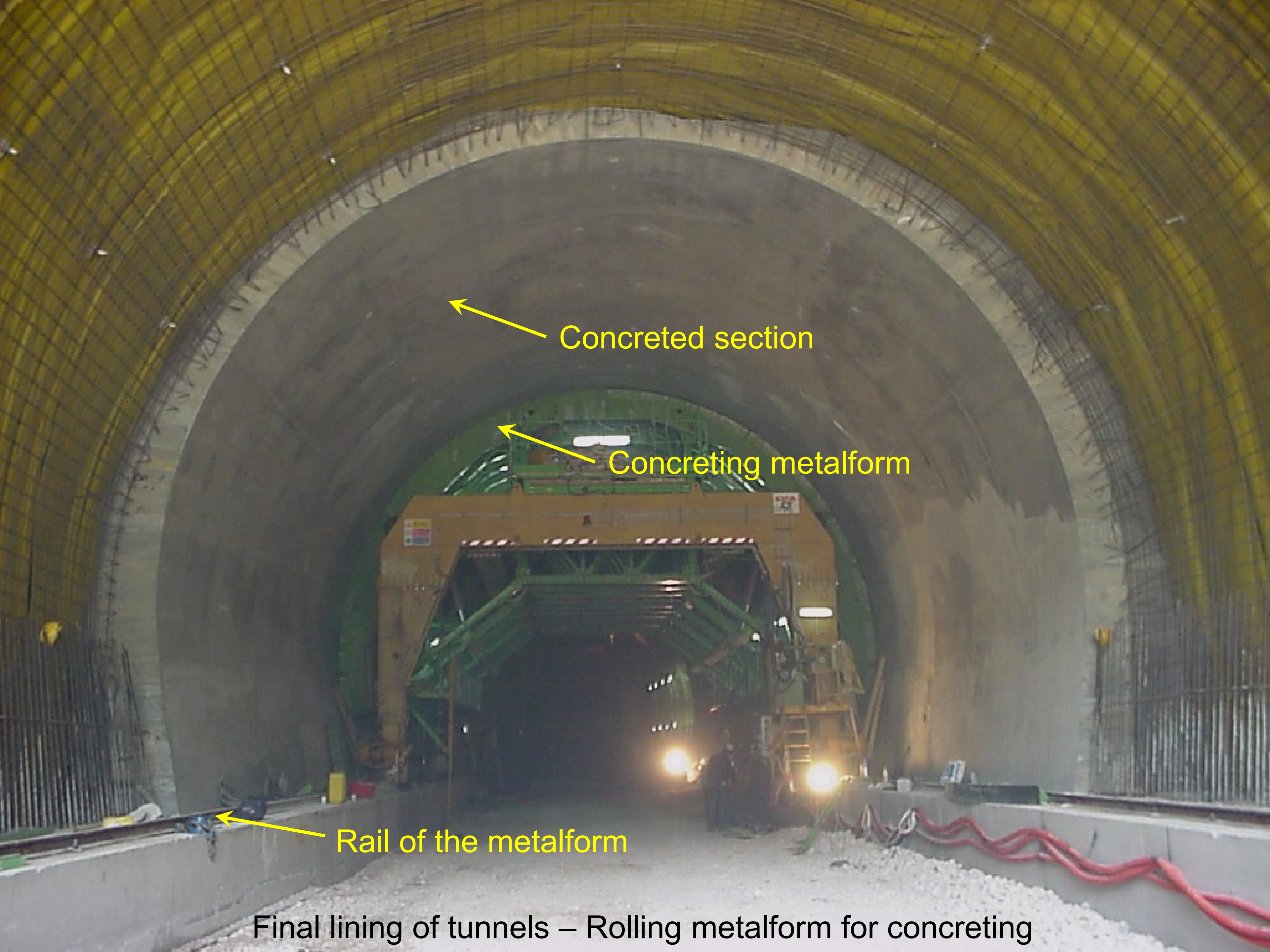
Final lining of tunnels – Erection of steel reinforcement

Final lining of tunnels – Rolling metal-form for concreting



Final lining of tunnels – Rolling metal form for concreting and erection platform of steel reinforcement

Final lining of tunnels – Rolling metal form for concreting and erection platform of steel reinforcement



Final lining of tunnels – Rolling metalform for concreting

Design of the final lining

1. Loads / actions

2. Design Code requirements
3. Specific requirements

- From surrounding ground ⇐ Most important actions
- Water pressures (external or internal)
- Surface structures (existing and future)
- Seismic actions
- Thermal / temperature changes
- Vehicle collision, explosion, fire, other operational actions

Final lining of tunnels

Loads from the surrounding ground are not "known" a priori: they are caused by the interaction between ground, primary support and final support (delayed installation and less stiff support → smaller loads)

So, the final lining cannot be designed with ground loads from a Specification, as is the case with other types of structural loads.

The expected range of loads on support is very wide (especially in deep tunnels):

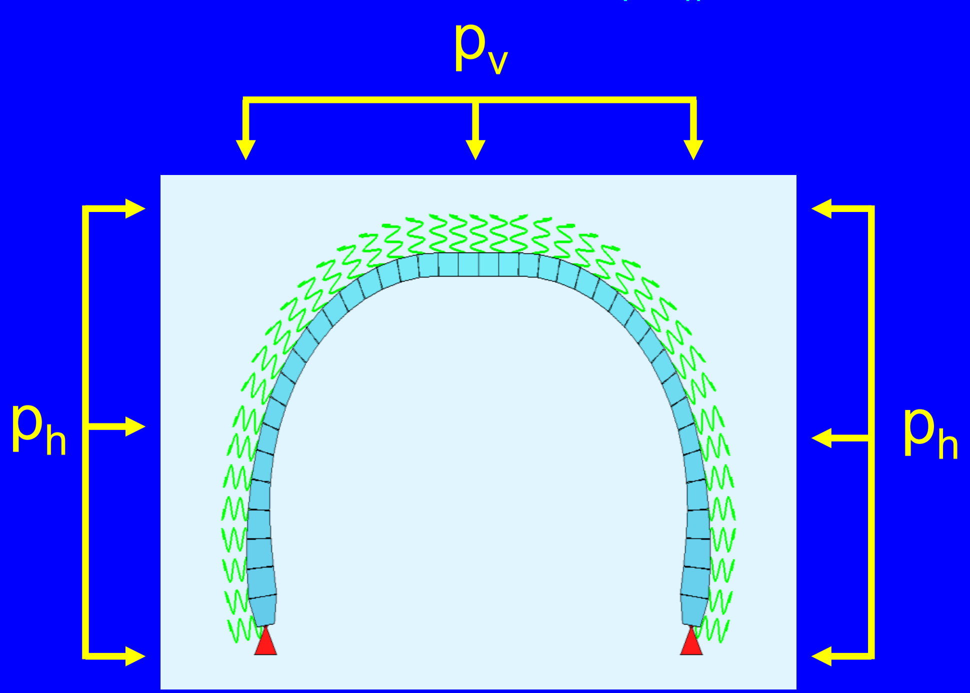
- Upper bound: The initial geostatic stresses ($p_v = \gamma H$, $p_h = K_o\ p_v$)
  since it can be claimed that in the long term, ground arching will be eliminated by creep and stresses will become geostatic (time depending on the creep characteristics of the ground).
  e.g. Tunnel at depth 150m : p_v = 24 kN/m^3 x 150m = 3600 kPa

- Lower bound: Zero (p = 0)
  since it can be claimed that ground loads are undertaken by the primary support and primary support is in equilibrium (its loads and deformations do not vary with time in common non-creeping soils). Thus, the final lining wil remain stress free.

- Reality: In between upper and lower bound
  depending mainly on the time of support installation, creep characteristics of the ground, and support stiffness.
  Note: Creep characteristics of the ground are not easy to measure or estimate. Thus, there is appreciable uncertainty in the magnitude of the loads acting on the final lining.

Final lining of tunnels – Analysis methods

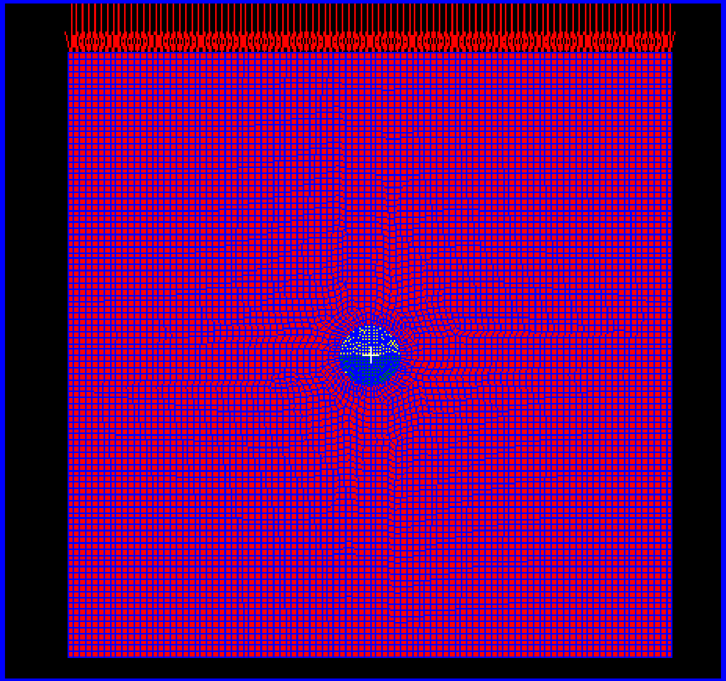
1. Use "pre-defined" loads (p_v, p_h)



Loads p_v, p_h are estimated by:

- Empirical methods (e.g. Terzaghi loads)
- Analytical methods (e.g. Terzaghi theory)
- Use same loads as on the primary lining
- Ground load of the plastic zone
- Full overburden load
- Local experience of the designer

2. Use loads resulting from analysis (usually FEM) of the interaction between ground and support

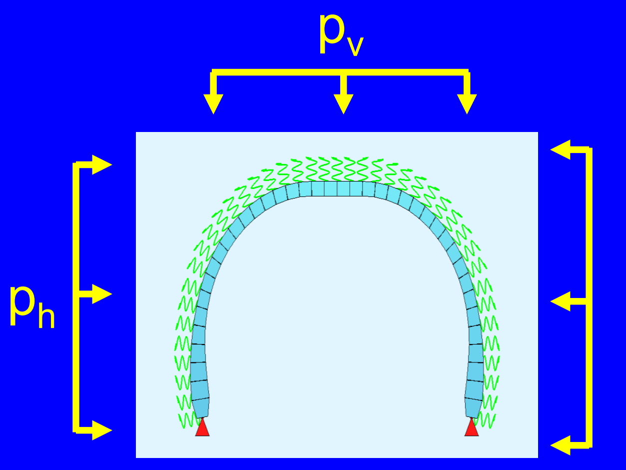


Analysis of the construction sequence: excavation, primary support and final support – requires models for ground creep and assumptions for transfer of loads from the primary support

Final lining of tunnels – Analysis methods

1. Analysis with known loads

1.1. Terzaghi (1946) empirical method, with modification by Deere (1970)



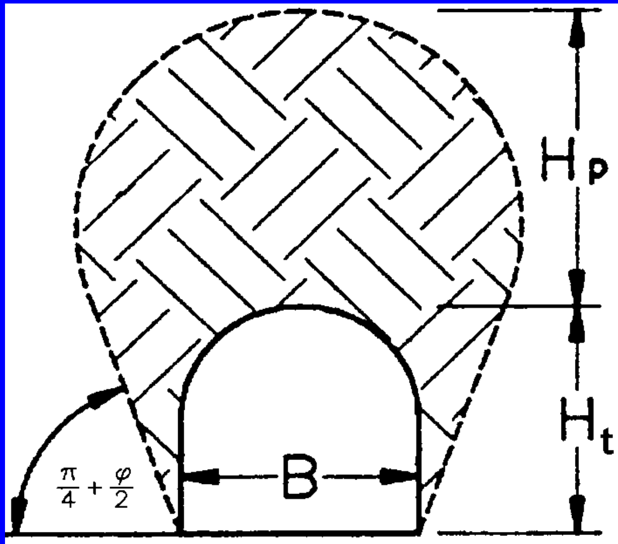
Tunnel width B and height H_t

H_p = rockmass height loading the tunnel

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 $(B + H_t)$

$$p_v = \gamma H_p$$

$$p_h \approx 0.5\, p_v$$



Note: The above loads are often combined with radial and circumferential Winkler springs on the tunnel lining

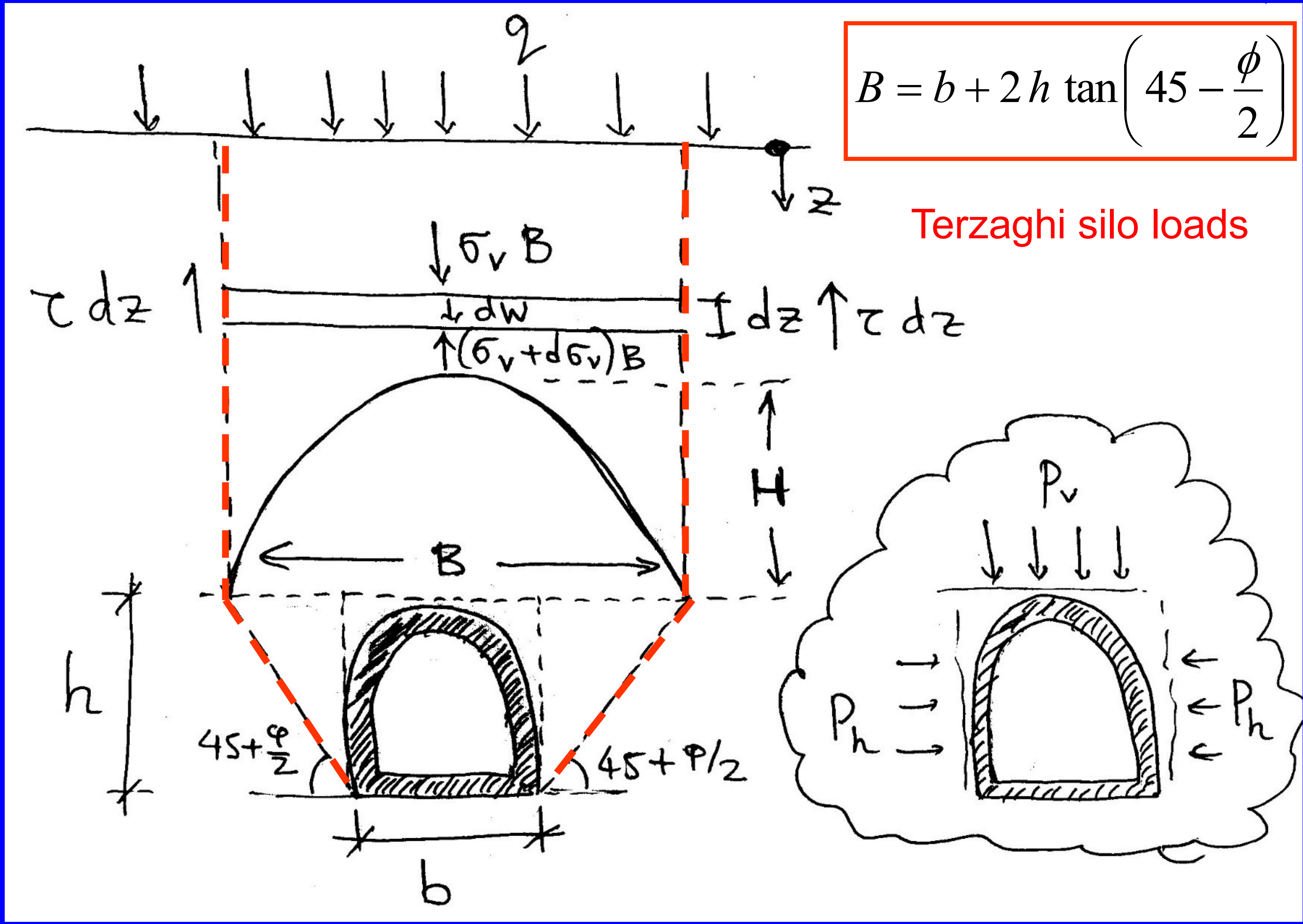
Final lining of tunnels – Analysis methods

1. Analysis with known loads

1.2. Terzaghi analytical method (silo loads)

σ_v = silo load

$\sigma_h = K\,\sigma_v$

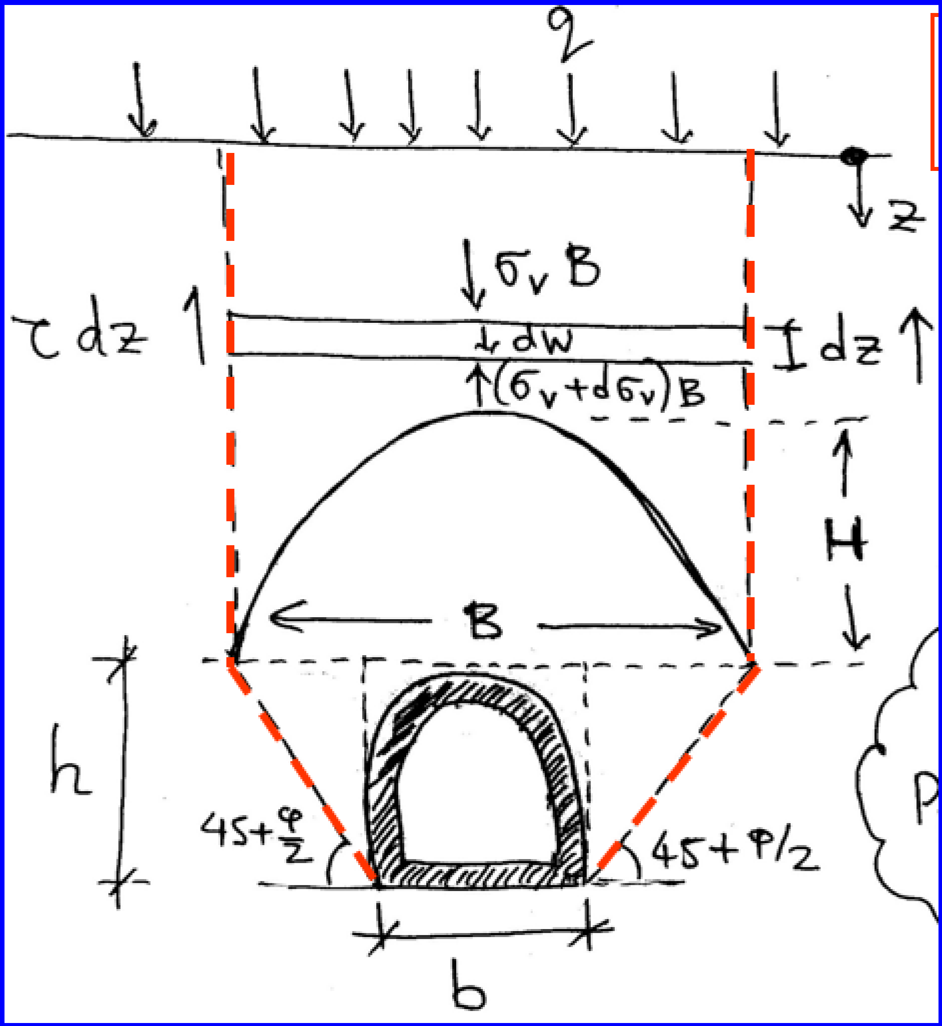


$$B = b + 2h\tan\left(45 - \frac{\phi}{2}\right)$$

Terzaghi silo loads

Final lining of tunnels – Analysis methods

1.2. Terzaghi analytical method (silo loads)



Force equilibrium in zone dz :

$$B\,\gamma\,dz + B\sigma_v = B(\sigma_v + d\sigma_v) + 2\tau\,dz$$

Shear stresses at the sides of zone dz:

$$\tau = c + \sigma_h \tan\phi = c + K\sigma_v \tan\phi$$

Combination of the above gives:

$$\frac{d\sigma_v}{dz} + \left(\frac{2K\tan\phi}{B}\right)\sigma_v = \frac{1}{B}(B\gamma - 2c)$$

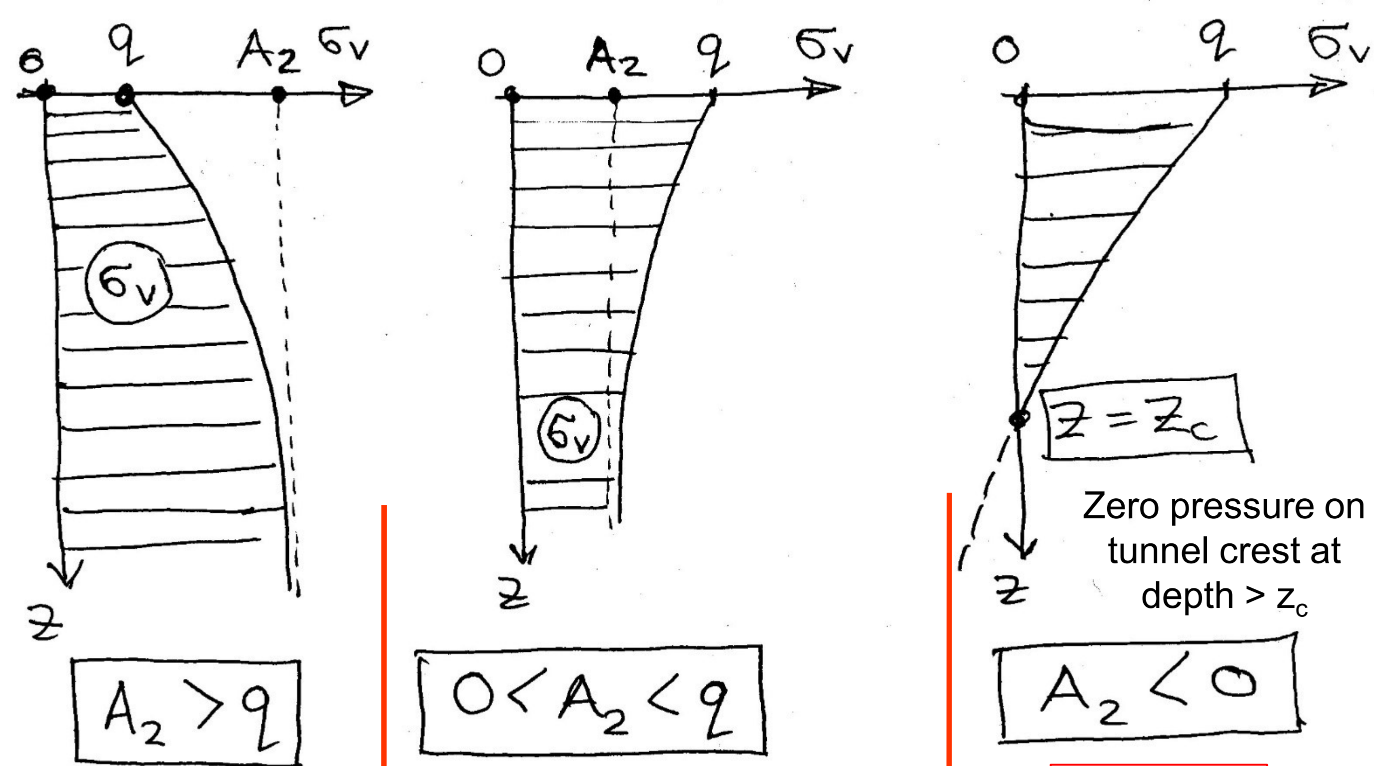
Solution of the differential equation with boundary condition: σ_v = q at z =0 gives the stress (σ_v) at each depth (z) above the tunnel (tunnel crest at z = H):

$$\sigma_v = A_2 + (q - A_2)e^{-A_1 z}$$

$$A_1 = \frac{2K\tan\phi}{B} \qquad A_2 = \frac{B\gamma - 2c}{2K\tan\phi}$$

1.2. Terzaghi analytical method (silo loads)

$$\sigma_v = A_2 + (q - A_2)e^{-A_1 z} \qquad A_1 = \frac{2K\tan\phi}{B} \qquad A_2 = \frac{B\gamma - 2c}{2K\tan\phi}$$



$$c < \frac{1}{2}B\gamma - Kq\tan\phi$$

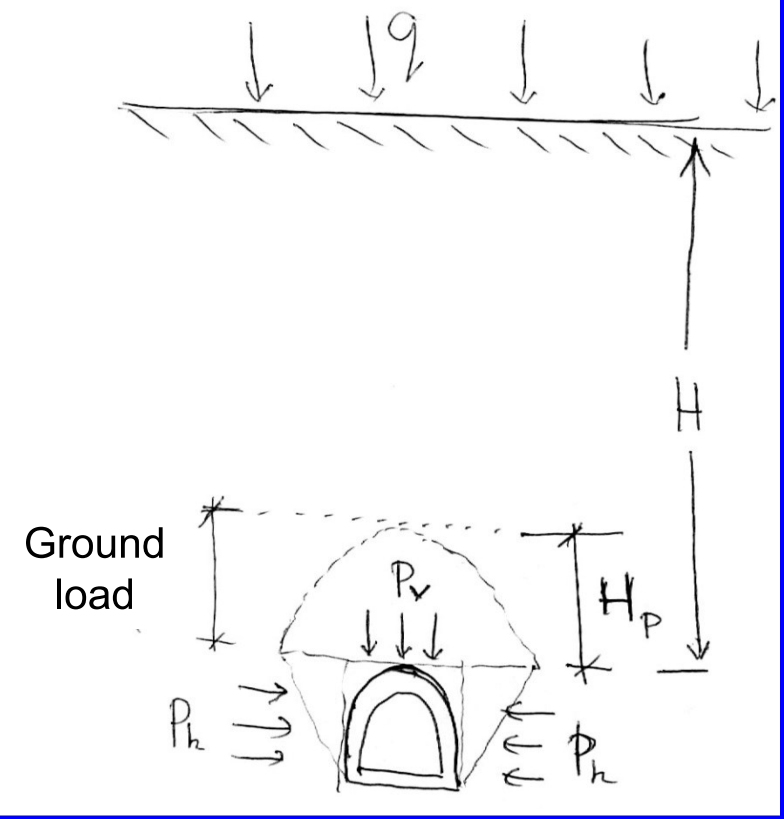
$$\frac{1}{2}B\gamma - Kq\tan\phi < c < \frac{1}{2}B\gamma$$

$$c > \frac{1}{2}B\gamma$$

Increasing cohesion (c)

$$z_c = \frac{B}{2K\tan\phi}\ln\left(1 + \frac{2Kq\tan\phi}{2c - \gamma B}\right)$$

Application of the Terzaghi analytical method for the calculation of ground pressures on the final lining



1. Calculate ground height (H_p) using Terzaghi empirical method (based on RQD).
2. In shallow tunnels (tunnel depth $H < H_p$), calculate pressures on final lining using the silo theory:

$$p_v = A_2 + (q - A_2) e^{-A_1 H}$$

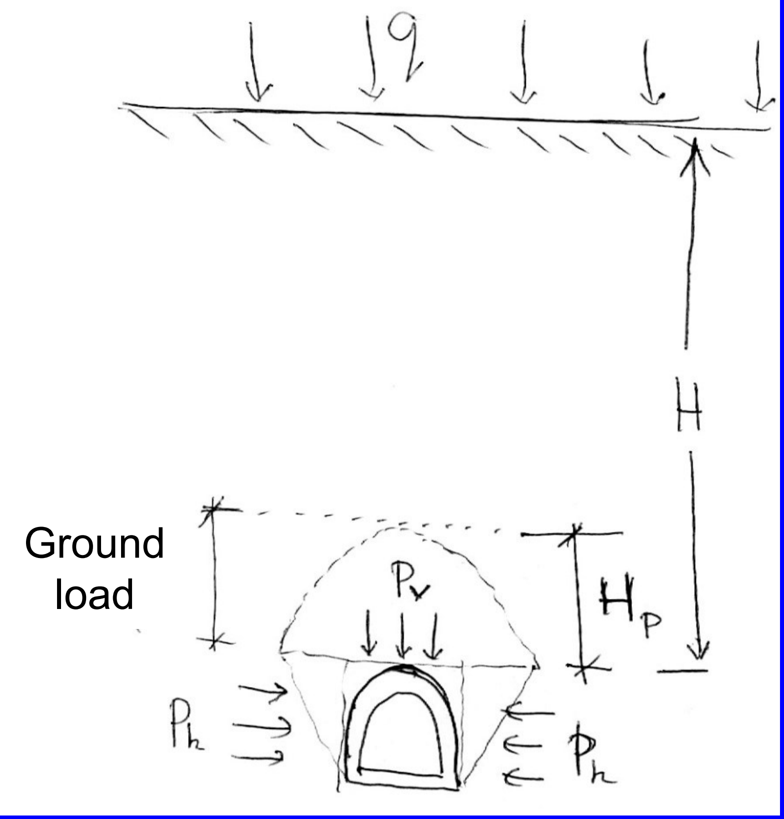
$$p_h \approx 0.5\, p_v$$

$$A_1 = \frac{2K \tan\phi}{B} \qquad A_2 = \frac{B\gamma - 2c}{2K \tan\phi}$$

3. In deep tunnels (tunnel depth $H > H_p$) calculate pressures on final lining using the silo theory with $z = H_p$ and zero q :

$$p_v = A_2 \left(1 - e^{-A_1 H_p}\right) \qquad p_h \approx 0.5\, p_v$$

Application of the Terzaghi analytical method for the calculation of ground pressures on the final lining



Example: Tunnel width b=10m, height h=10m

Rockmass with RQD = 15% :

H_p = 1.0 (b+h) = 20m

Rockmass parameters:

c = 50 kPa, φ =34^o , γ=21 kN/m^3, K=0.75

Surface surcharge: q = 20 kPa

$$A_1 = \frac{2K\tan\phi}{B} \qquad A_2 = \frac{B\gamma - 2c}{2K\tan\phi}$$

Compute:

B = 20.6m A_1 = 0.049 , A_2 = 329.443

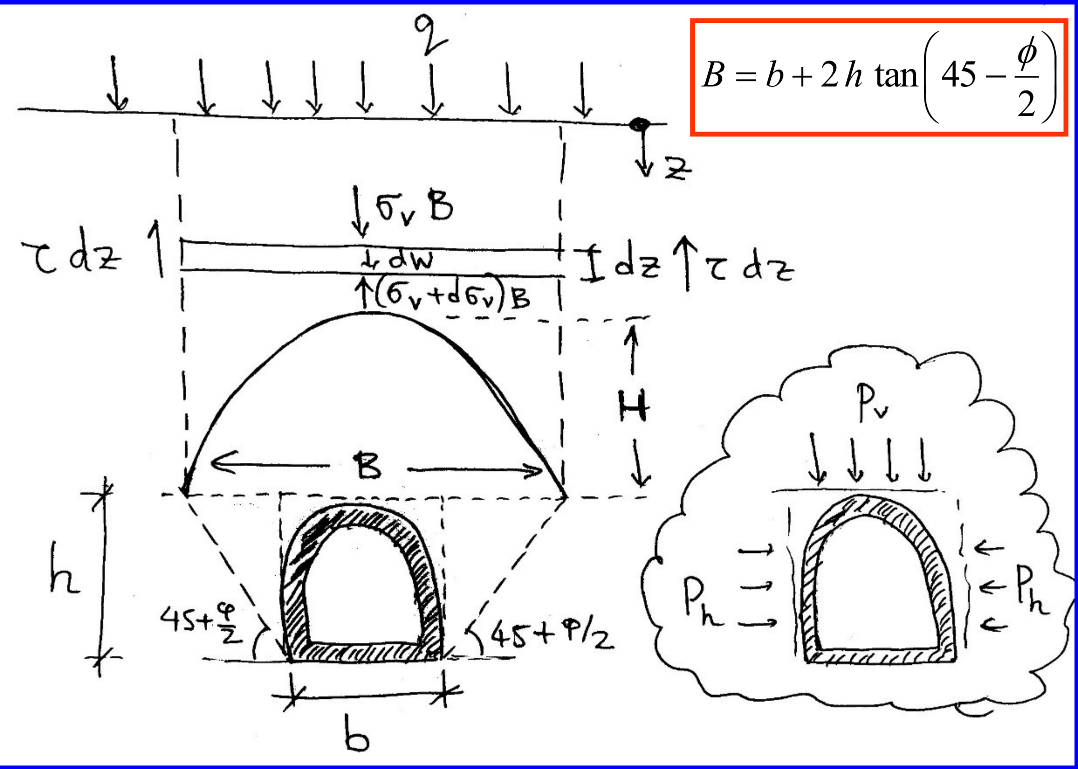
1. Deep tunnel, H=70m > H_p = 20m :

$$p_v = A_2\left(1 - e^{-A_1 H_p}\right) \quad \Rightarrow \ p_v = 206 \text{ kPa},\ p_h = 103 \text{ kPa}$$

2. Shallow tunnel, H=15m < H_p = 20m :

$$p_v = A_2 + (q - A_2)e^{-A_1 H} \quad \Rightarrow p_v = 181 \text{ kPa},\ p_h = 91 \text{ kPa}$$

1.3. Protodyakonov analytical method for deep tunnels



$$\boxed{B = b + 2h\tan\left(45 - \frac{\phi}{2}\right)}$$

Calculate the ground loading height (H) by the formula:

$$H = \frac{B}{2f}$$

where: $f = \tan\phi + \dfrac{c}{\sigma_{cm}}$

Typical values of (f) are given in the next slide

The ground mass loading the tunnel is assumed to be parabolic. Thus, its weight is:

$$W = \gamma S = \gamma\left(\frac{2}{3}BH\right) = \frac{1}{3f}\gamma B^2$$

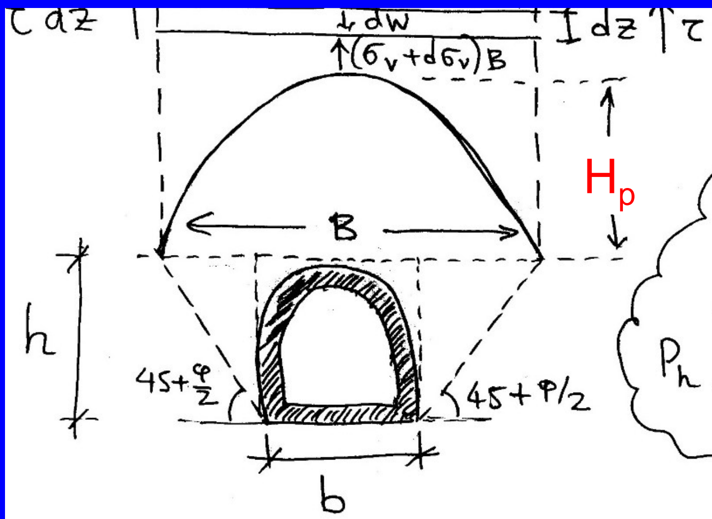
The average ground pressure on the tunnel is: $p_v = \dfrac{W}{B} \Rightarrow p_v = \dfrac{1}{3f}\gamma B$

$$p_h \approx 0.5\, p_v$$

1.3. Protodyakonov method

Typical values of f :

$$\boxed{p_v = \frac{1}{3f}\gamma B}$$



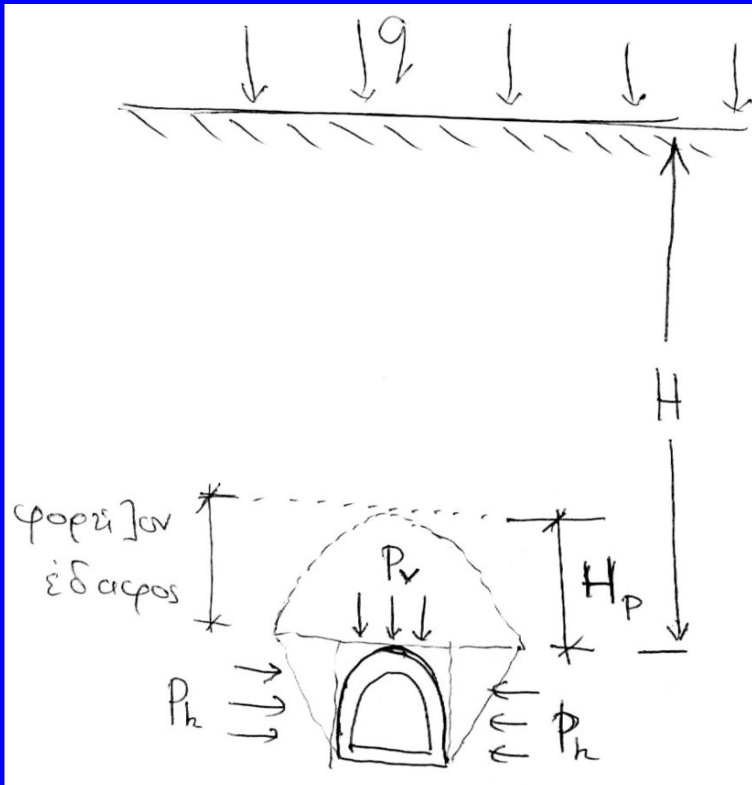
$$B = b + 2h\tan\left(45 - \frac{\phi}{2}\right)$$

$$H_p = 1.5\, p_v / \gamma$$

Strength Factors after Protodyakonov

Category	Strength grade	Denotation of rock (soil)	Unit weight (kg/m³)	Crushing strength σ_t (kg/cm²)	Strength factor f
I	Highest	Solid, dense quartzite, basalt and other solid rocks of exceptionally high strength	2800 3000	2000	20
II	Very high	Solid, granite, quartzporphyr, silica shale. Highly resistive sandstones and limestones	2600–2700	1500	15
III	High	Granite and alike. Very resistive sand- and limestones. Quartz. Solid conglomerates.	2500–2600	1000	10
IIIa	High	Limestone, weathered granite. Solid sandstone, marble. Pyrites.	2500	800	8
IV	Moderately strong	Normal sandstone	2400	600	6
IVa	Moderately strong	Sandstone shales	2300	500	5
V	Medium	Clay-shales. Sand- and limestones of smaller resistance. Loose conglomerates.	2400–2800	400	4
Va	Medium	Various shales and slates. Dense marl.	2400–2600	300	3
VI	Moderately loose	Loose shale and very loose limestone, gypsum, frozen ground. Common marl. Blocky sandstone, cemented gravel and boulders, stoney ground	2200–2600	200–150	2
VIa	Moderately loose	Gravelly ground. Blocky and fissured shale, compressed boulders and gravel, hard clay.	2200–2400	—	1·5
VII	Loose	Dense clay. Cohesive ballast. Clayey ground.	2000–2200	—	1·0
VIIa	Loose	Loose loam, loess, gravel.	1800–2000	—	0·8
VIII	Soils	Soil with vegetation, peat, soft loam, wet sand.	1600–1800	—	0·6
IX	Granular soils	Sand, fine gravel, upfill	1400–1600	—	0·5
X	Plastic soils	Silty ground, modified loess and other soils in liquid condition	—	—	0·3

1.3. Protodyakonov method



Example:

Tunnel width b=10m and height h=10m

Tunnel depth: H = 30m

Rockmass with f = 1 :

Ground parameters: $\varphi = 34^o$ $\gamma = 21 \ kN/m^3$

$$B = b + 2h \tan\left(45 - \frac{\phi}{2}\right) = 20.6 \text{ m}$$

$$H_p = \frac{B}{2f} = 10.3 \text{ m}$$

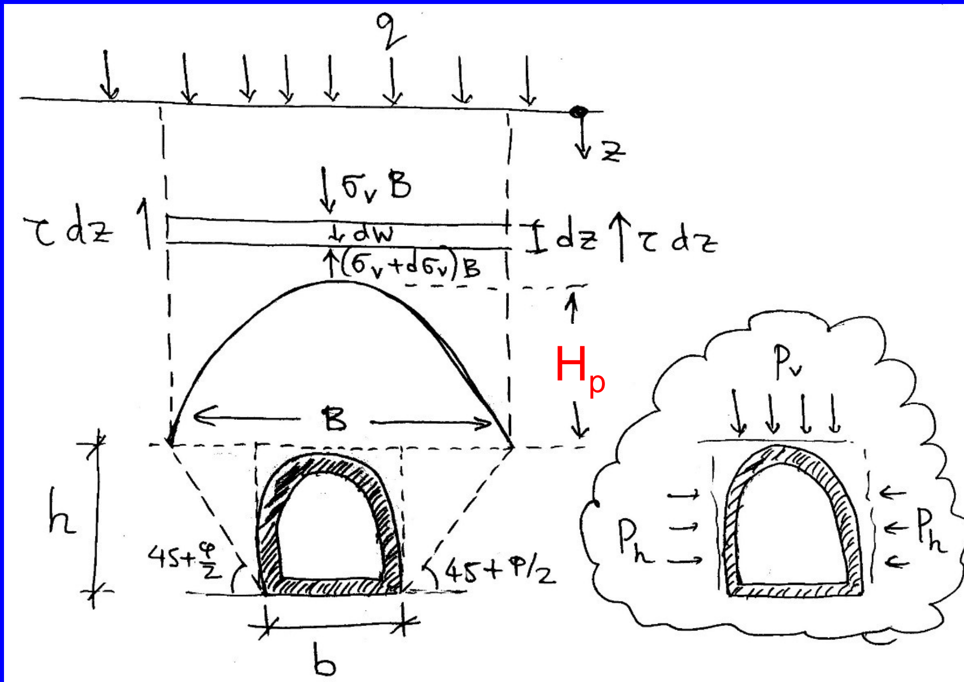
Shallow tunnel ($H < H_p$): use $H_p = H$

Deep tunnel ($H > H_p$) : Use calculated H_p

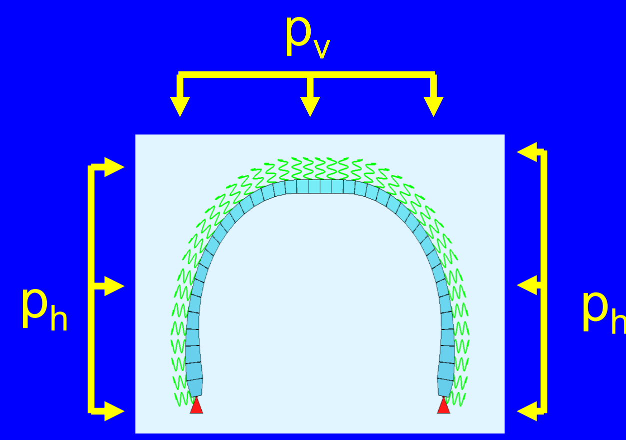
$p_v = (2/3)\ H_p\ \gamma = 0.67 \times 10.3 \times 21 = 144$ kPa

$p_h = 0.5 \times 144 = 72$ kPa

Note: Protodyakonov method usually gives lower pressures on the tunnel lining than the Terzaghi method.



LATERAL SPRING SUPPORTS



RADIAL SPRINGS: $$k_r = \frac{2E}{D(1+\nu)}$$

k_r is only compressive – if tensile (tunnel wall tends to move away from the ground, use k_r=0)

TANGENTIAL SPRINGS: $$k_\theta = (\tan\delta)\, k_r$$

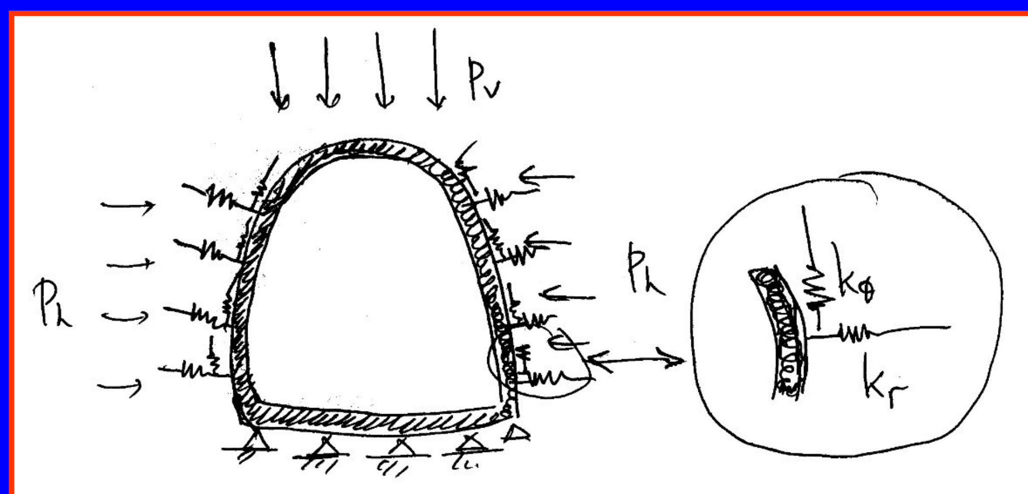
D = average tunnel width

E, v = ground modulus

δ = ground-support friction angle

δ = 5-10° with waterproof membrane

δ = 25-45°, without waterproofing membrane, depending on the ground roughness



Ground pressures on the final lining of tunnels

Internationally accepted methods to assess the ground pressures on the final lining differ significantly. There is also disagreement on the need for steel reinforcement (e.g. Germany → Yes, Austria → No).

Results of investigation by the US Dept. of Transportation (Design Recom. for Concrete Linings of Transportation Tunnels, 1983) on the methods used to design the Final Lining among 16 large US Tunnelling Consultants:

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1. Road tunnels in relatively good rock:

No	Method to design Final Lining
10	Minimum thickness and reinforcement (without calculations)
2	Use loads of expected wedge failures
3	Use Terzaghi loads, or other similar methods based on RQD
1	Use full geostatic loads (experience with shallow tunnels, where geostatic loads are relatively small)

Ground pressures on the final lining of tunnels

Results of investigation by the US Dept. of Transportation (Design Recom. for Concrete Linings of Transportation Tunnels, 1983) on the methods used to design the Final Lining among 16 large US Tunnelling Consultants:

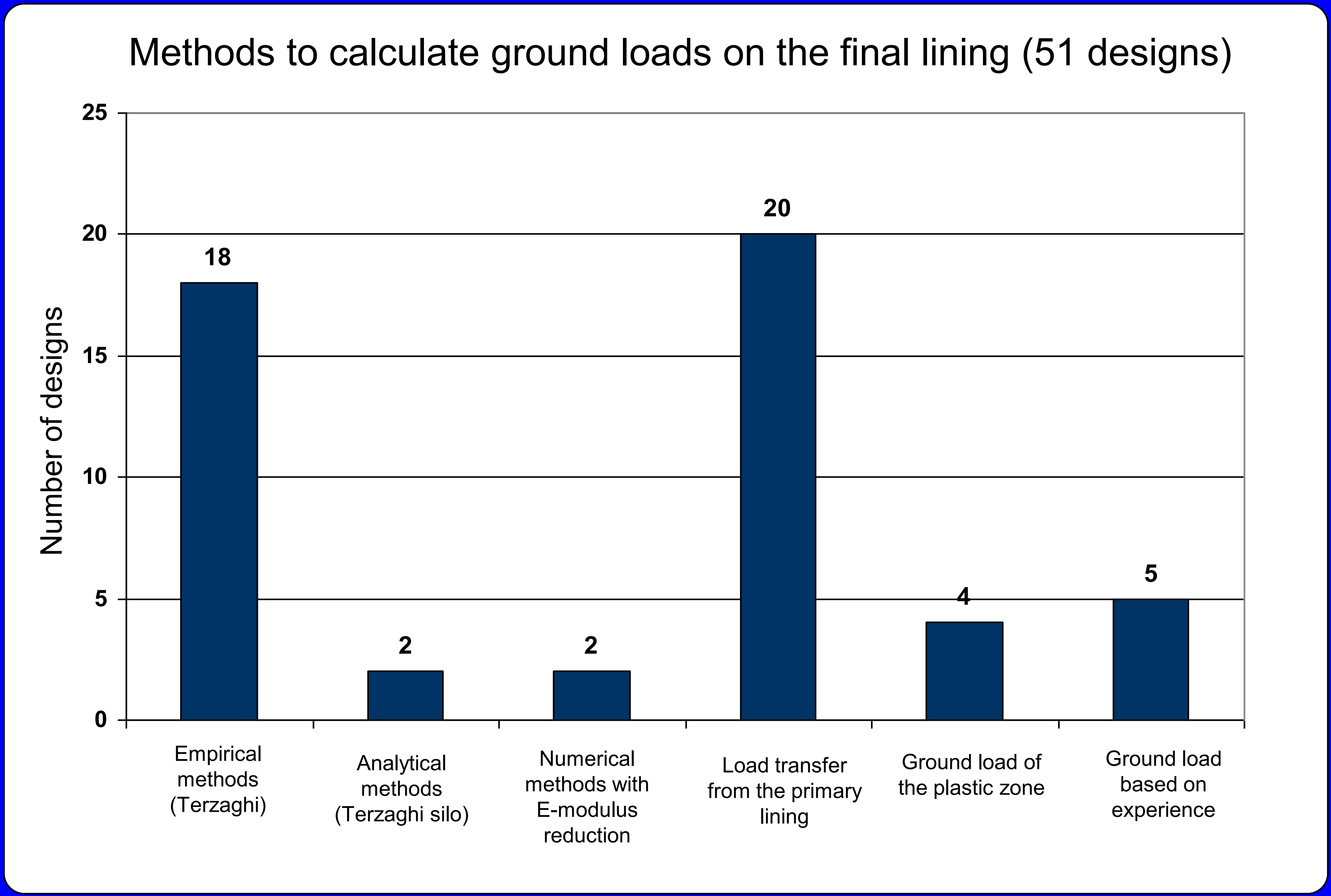
2. Road tunnels in weak and very weak rocks: (only 6 responded)

No	Vertical load	Horizontal load
2	Minimum thickness and reinforcement (without calculations)	
1	Ground load: H = (1.5 ÷ 2.0) width	60% of vertical + Winkler springs
1	• Full geostatic load for depth H < 25m • Gradually reduced geostatic load for depth H>25m, with maximum the Terzaghi loads	Intermediate between K_o and passive pressure
1	Full geostatic loads	87.5% of vertical
1	Full geostatic loads	K_o times vertical

Ground pressures on the final lining of tunnels

Collection of data from tunnels of the Egnatia Highway (Fortsakis, 2007)

37 twin tunnels, 166 typical sections, 38 designs, 19 design offices



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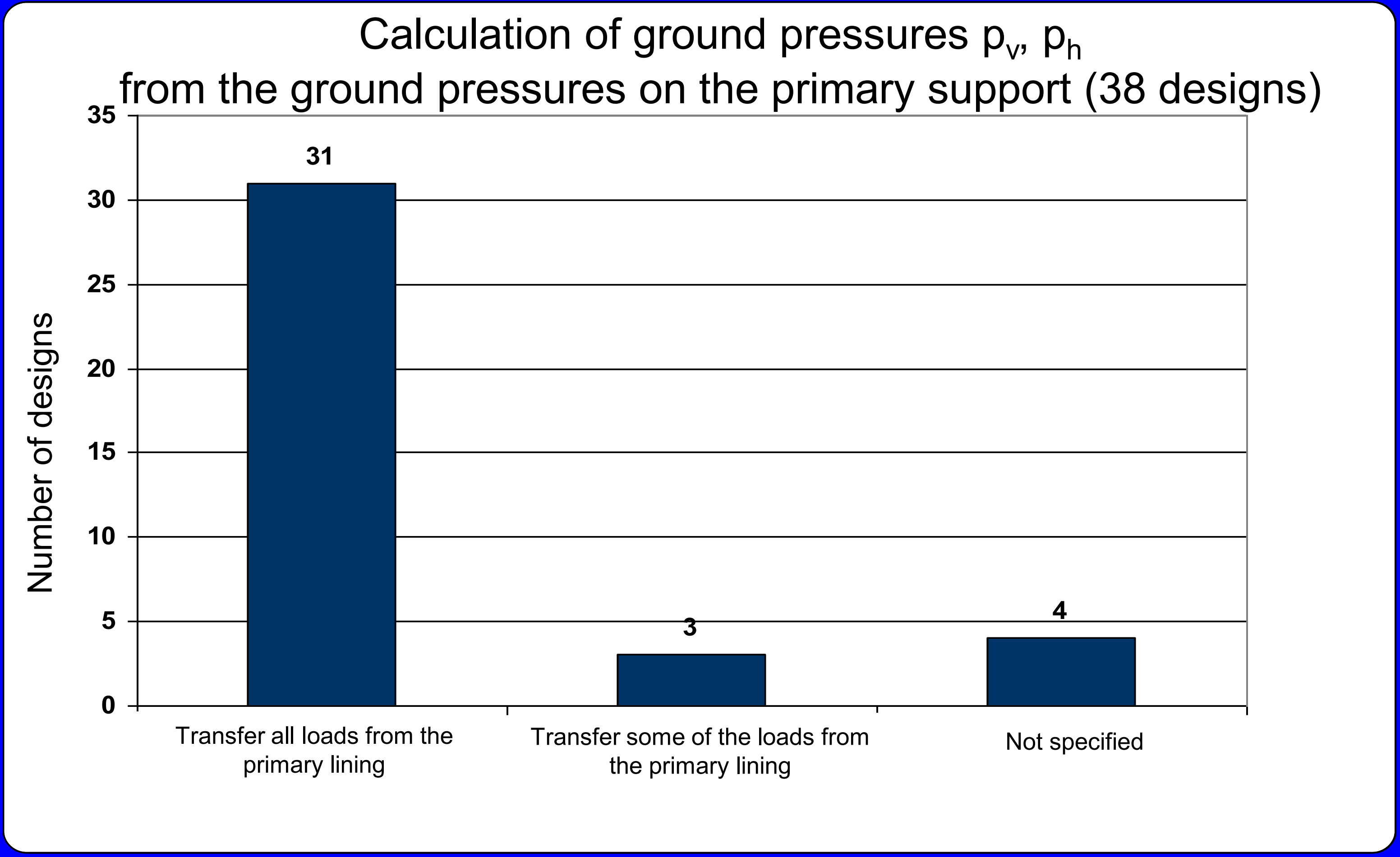

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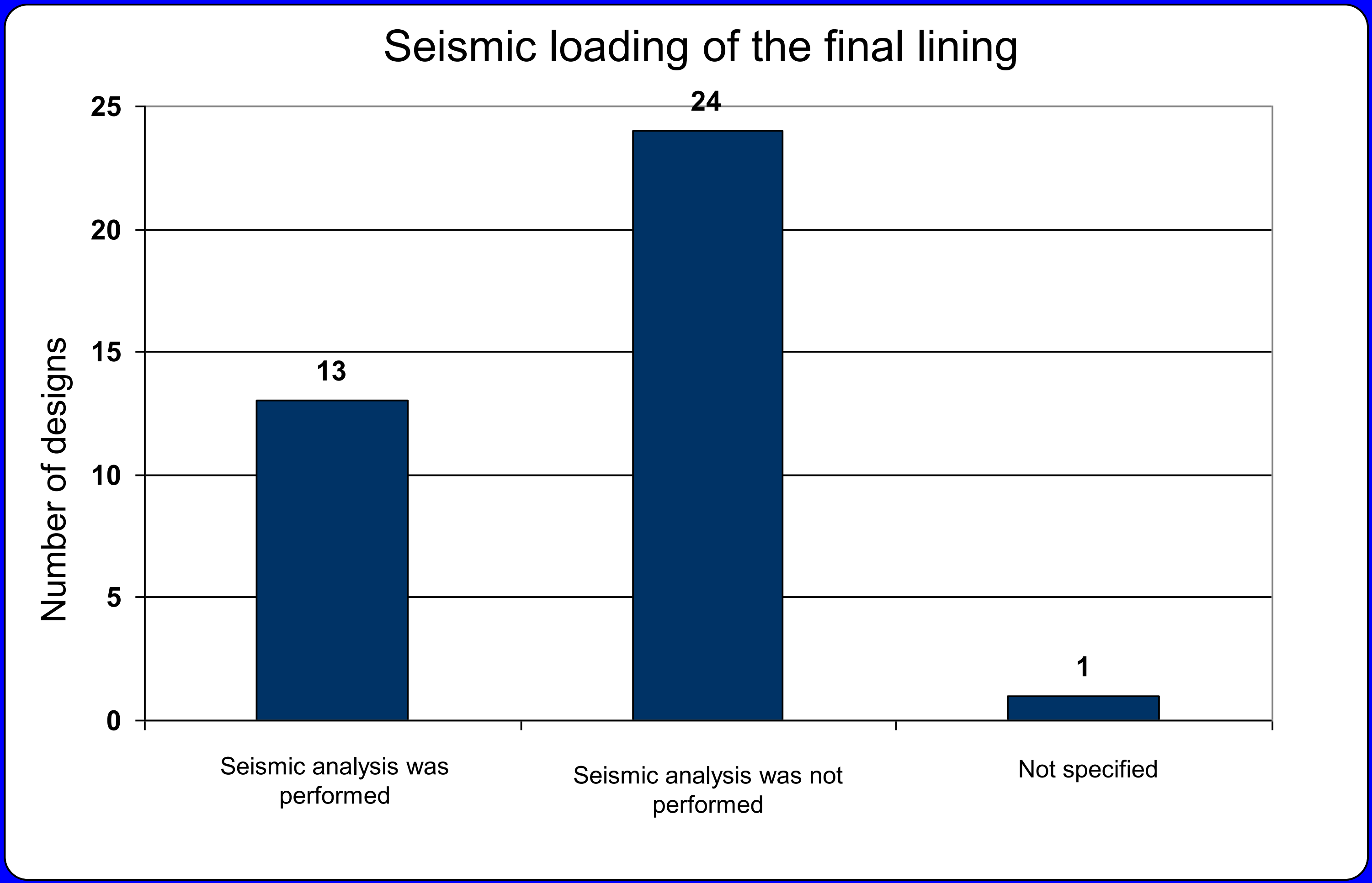


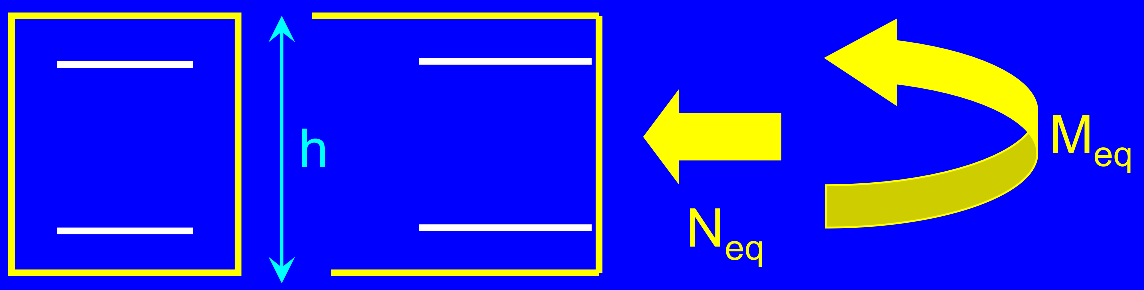
Ground pressures on the final lining of tunnels

Collection of data from tunnels of the Egnatia Highway (Fortsakis, 2007)

Correlation of the final lining capacity ($M_{eq,tot}$) with ground conditions (σ_{cm} / p_o)

A measure of the total lining bending capacity ($M_{eq,tot}$) was calculated as follows:

Using the thickness of the final lining (h) and the amount of steel reinforcement (fraction μ of the concrete volume), the axial and bending capacities (N_{eq}, M_{eq}) of the final lining we computed using standard RC formulae:



A measure of the total bending capacity of the lining ($M_{eq,tot}$) was calculated by the formula:

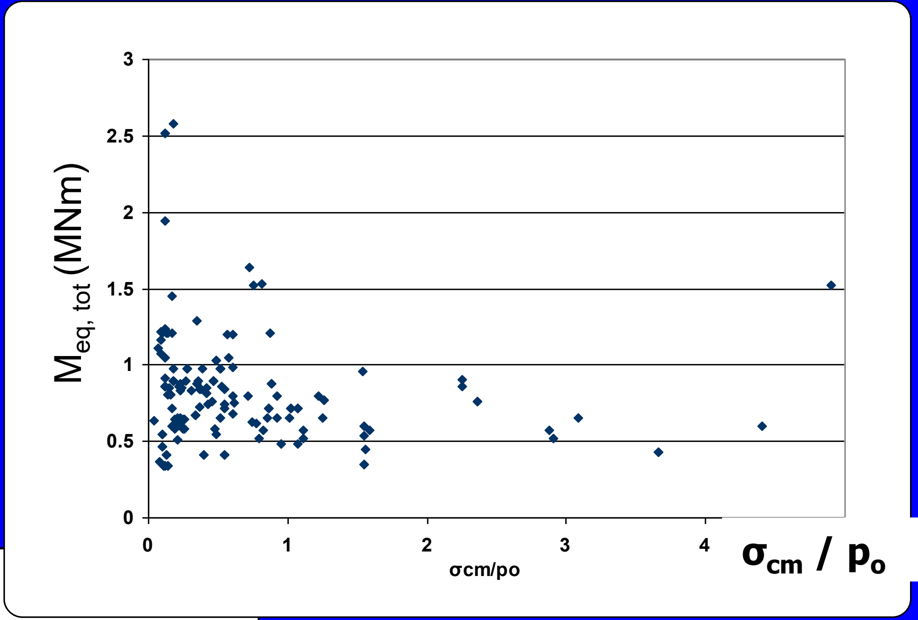
$$\boxed{M_{eq,tot} = N_{eq}\left(\frac{h}{8}\right) + M_{eq}}$$

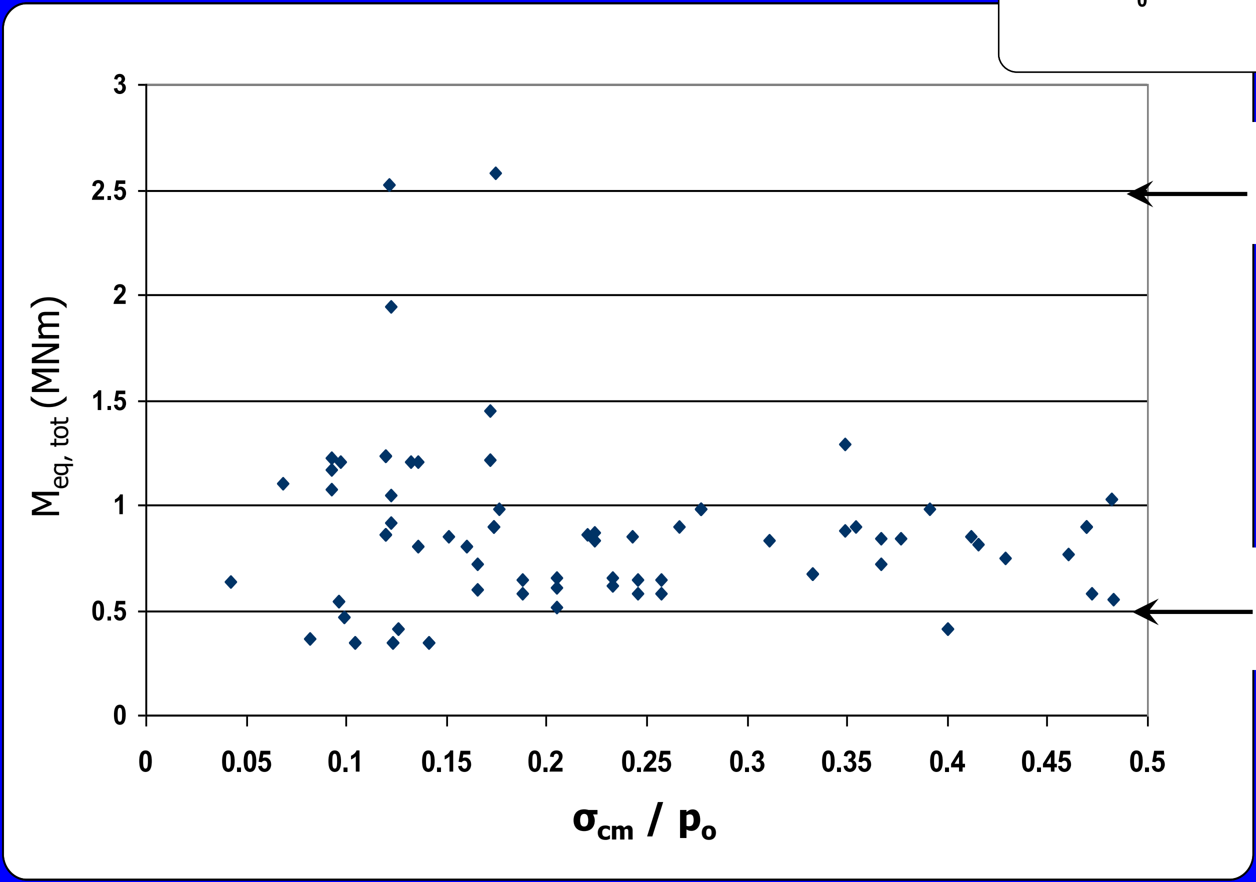
Data from tunnels of the Egnatia Highway (Fortsakis, 2007)

Correlation of total lining bending capacity ($M_{eq,tot}$) with ground conditions (σ_{cm} / p_o)

$$M_{eq,tot} = N_{eq}\left(\frac{h}{8}\right) + M_{eq}$$

Correlation is <u>very poor</u> (final lining capacity is independent of ground conditions)





Final ling thickness: 60 cm
Reinforcement: 3700 kg / m

Final ling thickness: 30 cm
Reinforcement: 590 kg / m

Analysis of ground loads on the final lining of tunnels

1. Loads from rock-bolts and steel sets

   Due to gradual loss of tension (and erosion) of rock bolts and erosion of steel sets (they do not have the required concrete cover)

2. Loads from the shotcrete of the primary support

   (a) Due to difference in the safety factor between temporary (primary support) loads and permanent (final support) loads.

   (b) Due to larger creep of the shotcrete compared to the final lining (shotcrete has high tendency for creep).

"Easy" loads

---

3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground <u>squeezing</u>

   Ground creep cause a tendency for inward convergence of the tunnel wall. When this tendency is obstructed by a tunnel lining (primary and/or final), the lining exerts a pressure on the ground to prevent the tendency for ground convergence (proportional to the tendency).

4. Loads due to ground <u>swelling</u> caused by negative ground consolidation

   Ground stress reduction due to tunnel excavation, causes negative pore water pressures. Gradual diffusion of these pressures requires water to be sucked from the surrounding causing ground swelling. When ground swelling is obstructed by a tunnel lining (primary and/or final), the lining exerts a pressure on the ground to prevent this tendency.

"Difficult" loads

Origin of ground loads on the final lining of tunnels

1a. Loads from rock-bolts:

Gradual unloading due to ground creep (at the grout-ground interface) and eventual corrosion (uncontrolled grout cover).

Result: The total of rock bolt loads is transferred to the final lining, as equivalent pressure p

---

Common passive rockbolts – capacity 200 kN (Φ25mm - S500)

Grid spacing (a x b)	Equiv. pressure p (kPa)	p / p_o for a tunnel depth H=150m ($p_o = \gamma H$)
2.5 x 2.5 m	25	$p = 0.007\ p_o$
2.0 x 2.0 m	40	$p = 0.011\ p_o$
1.5 x 1.5 m	70	$p = 0.019\ p_o$
1.0 x 1.0 m	160	$p = 0.044\ p_o$

Note: $P_{all} = 160$ kN (FS = 1.25) , $p = P_{all} / a\ b$, $p_o = \gamma H = 3.6$ MPa

Origin of ground loads on the final lining of tunnels

1b. Loads from steel sets:

Gradual corrosion due to uncontrolled concrete cover.

Result: The total of steel set loads is transferred to the final lining, as equivalent pressure p

---

Steel set HEB 140 – Fe360 ($A=43cm^2$) in a tunnel width B=12m

Set distance s (m)	Equiv. pressure p (kPa)	p / p_o for a tunnel depth H=150m ($p_o = \gamma H$)
2.0	42	$p = 0.012\ p_o$
1.5	55	$p = 0.015\ p_o$
1.0	83	$p = 0.023\ p_o$
0.75	111	$p = 0.031\ p_o$

Note: P_{all} = 500 kN (FS = 2) , $p = P_{all} / (B/2)\ s$, $p_o = \gamma H$ = 3.6 MPa

Origin of ground loads on the final lining of tunnels

2a. Loads from the shotcrete of the primary support due to SF:

Due to safety factor (SF) difference between temp and perm support

e.g. Temp support: SF = 1.50   Perm support: SF = 1.75

Result: Part of the load taken by the temp support (shotcrete) is logistically transferred to the final lining as an equivalent pressure p

Shotcrete C20/25 in a tunnel width B=12m

Shotcrete thickness t (cm)	Equiv. pressure p (kPa)	p / p_o for a tunnel depth H=150m ($p_o = \gamma H$)
10	40	$p = 0.011\ p_o$
15	60	$p = 0.017\ p_o$
20	80	$p = 0.022\ p_o$
25	100	$p = 0.028\ p_o$

Note: $\Delta\sigma_{all} = (\sigma_y/SF_1) - (\sigma_y/SF_2) = 25/1.50 - 25/1.75 = 2.38$ MPa

$p = \Delta\sigma_{all}\ t / (B/2)$, $p_o = \gamma H = 3.6$ MPa

Initial ground pressure by shotcrete: $p_1 = (\sigma_y/SF_1)\, t / (B/2) = 280 \div 690$ kPa

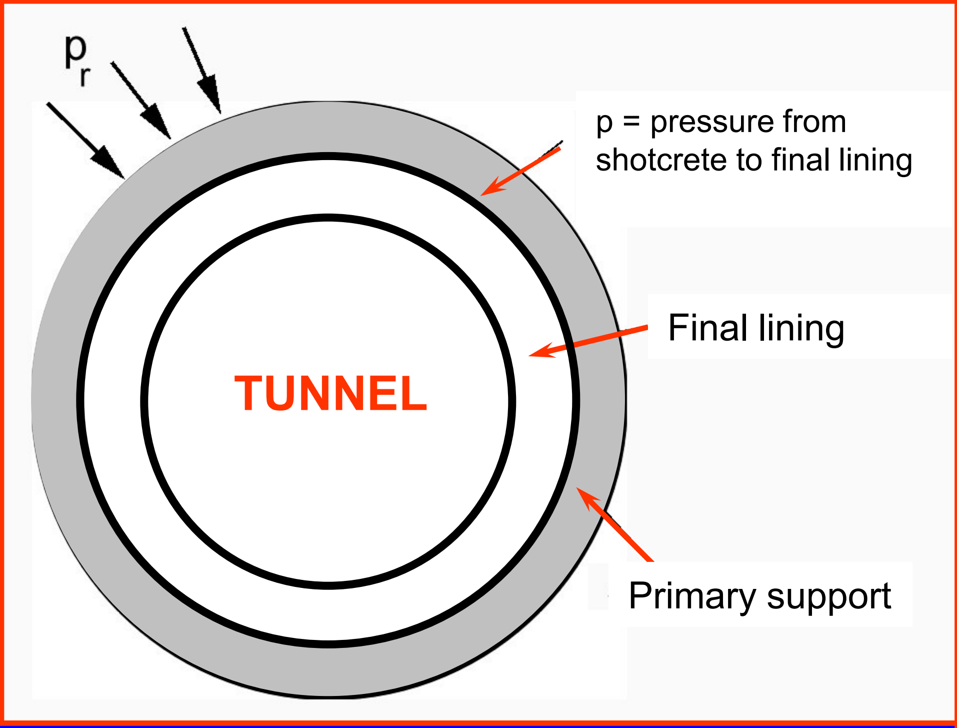
$p/p_1 \approx 15\%$ of initial shotcrete pressure is transferred to final lining

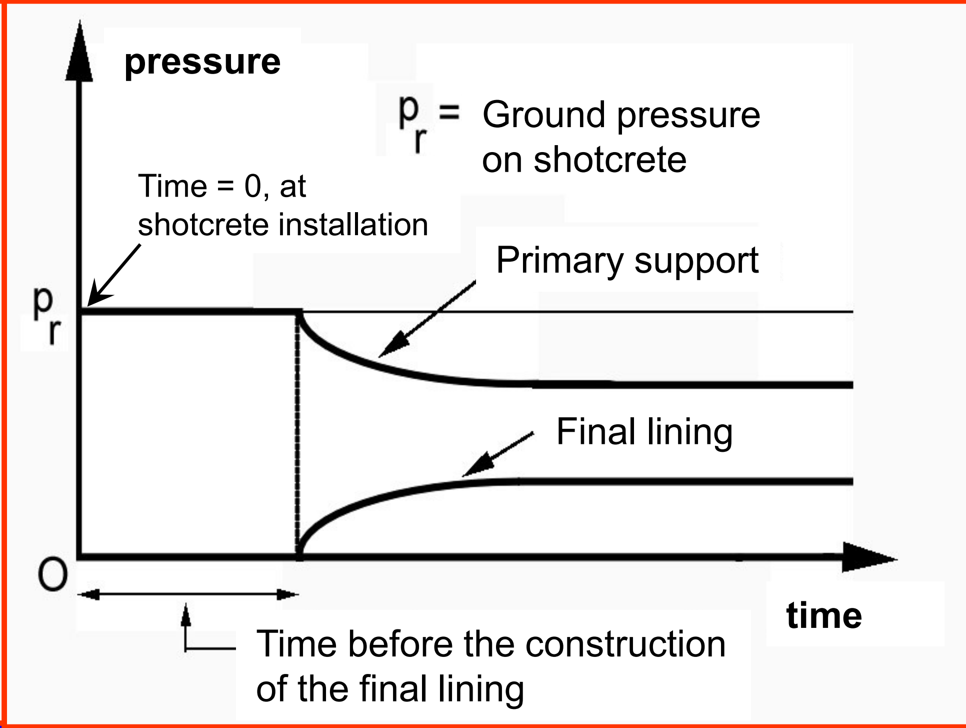
2b. Loads from the shotcrete of the primary support due to creep:

Due to larger creep of the shotcrete compared to the creep of the final lining (without ground creep)

- Shotcrete: Intensely loaded (high creep), but with larger age (low creep)
- Final lining: Initially unloaded (low creep) and young in age (higher creep)

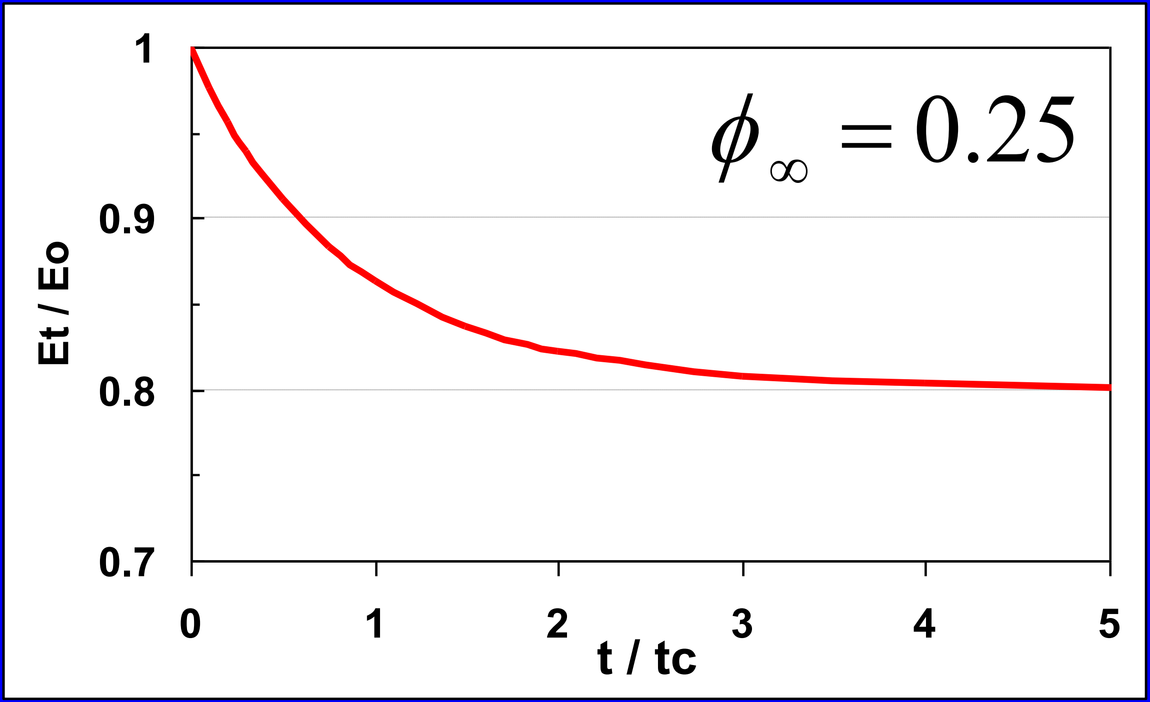
Result: Part of the shotcrete load is transferred to the final lining as an equivalent pressure p





2b. Loads from the shotcrete of the primary support due to creep:

Creep model for concrete: $$E_t = \frac{E_o}{1+\phi_\infty\left[1-\exp\left(-t/t_c\right)\right]}$$



$$E_{t\infty} = \frac{E_o}{1+\phi_\infty}$$

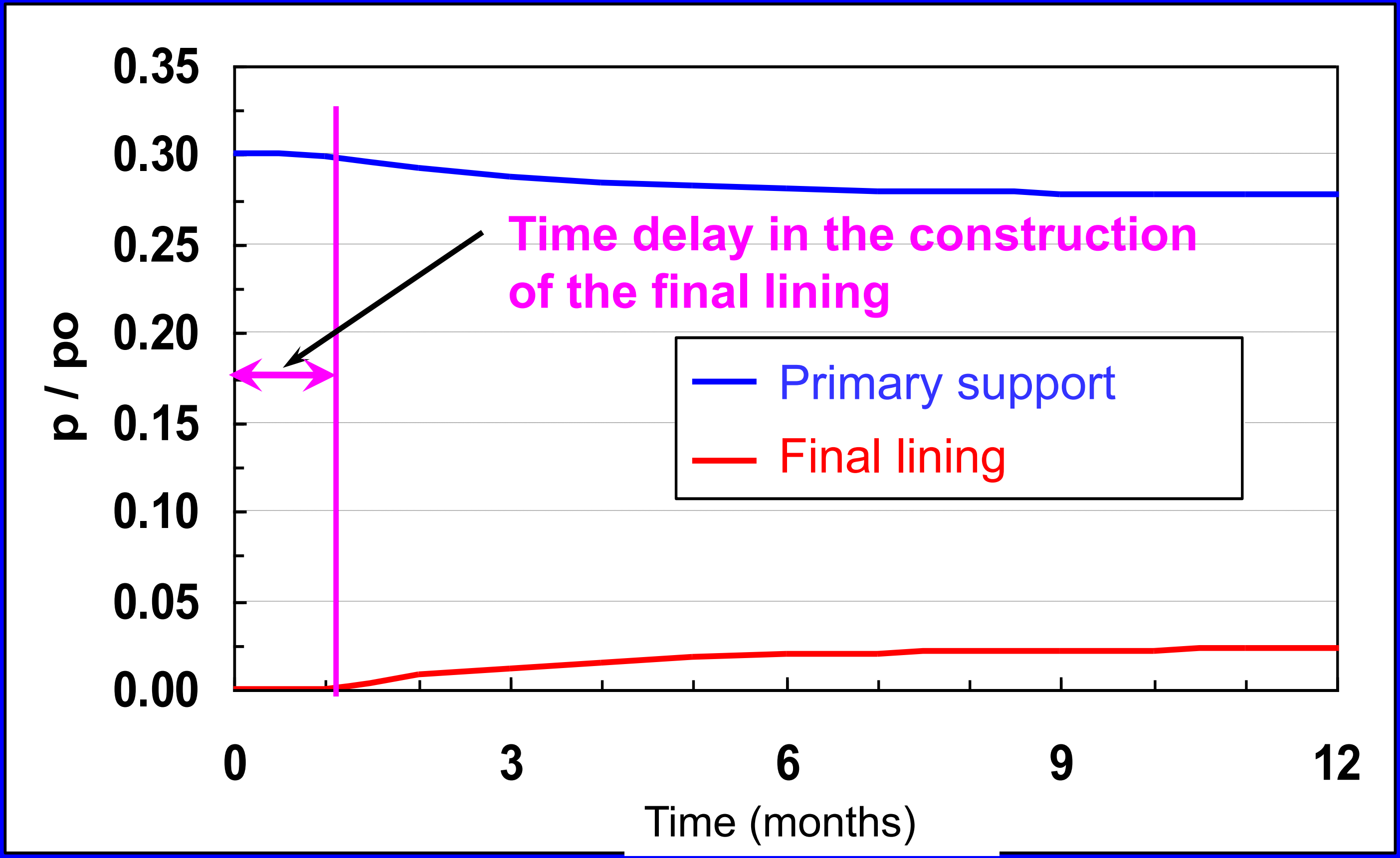
$$\phi_\infty = \frac{\varepsilon_{t\infty}}{\varepsilon_e} = \frac{\text{Creep strain}}{\text{Elastic strain}}$$

	E_o (GPa)	$E_{t\infty}/E_o$	ϕ_∞	t_c (months)
Shotcrete	30	0.80	0.25	3
Cast insitu concrete	30	0.90	0.11	12

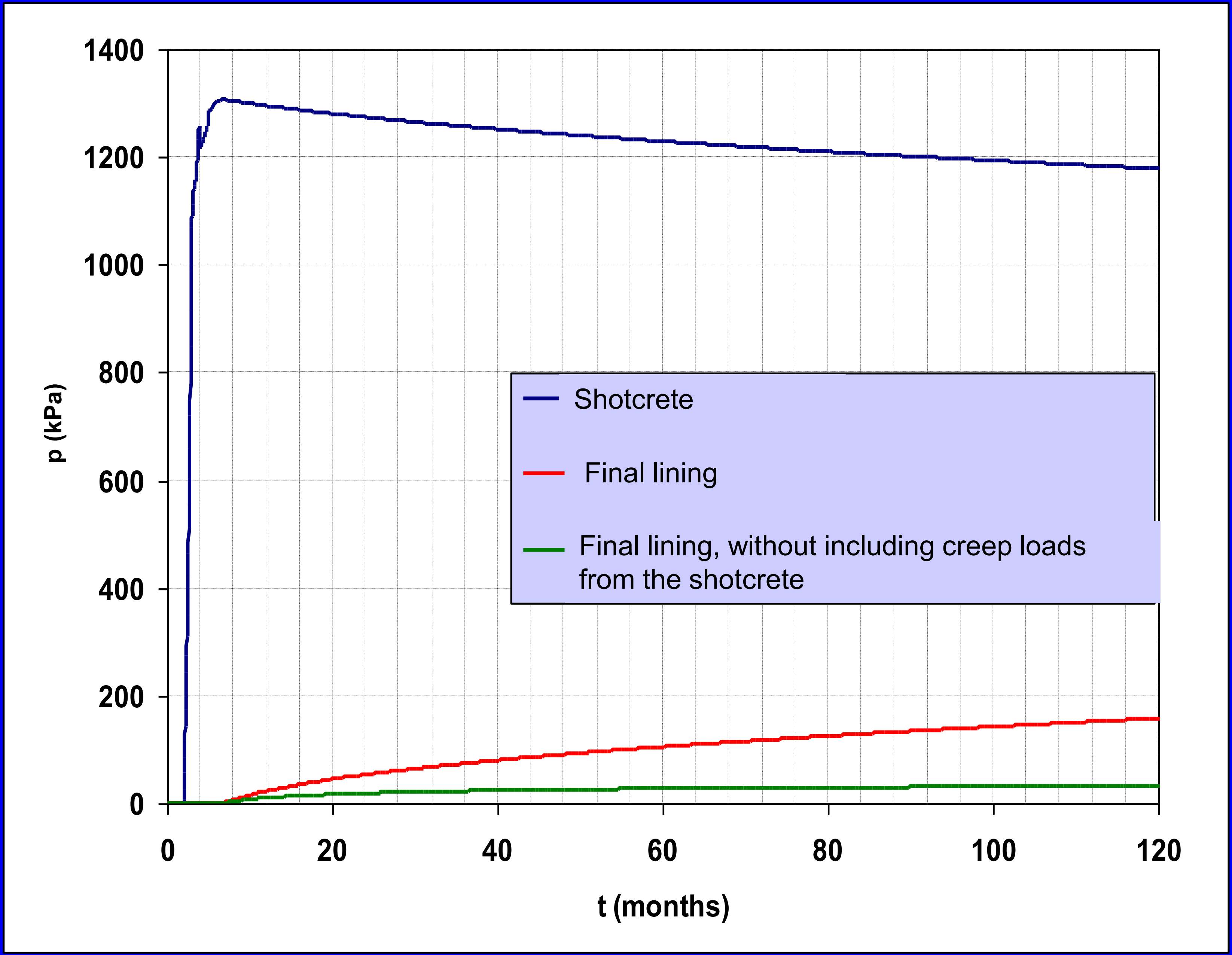
2b. Loads from the shotcrete of the primary support due to creep:

Problem parameters:

- Creep characteristics of shotcrete and final lining
- Thickness ratio (final / primary): t_M / t_A
- Ground deconfinement (at tunnel excavation): $\lambda = 1 - p_r / p_o$
- *Time delay in the construction of the final lining (Δt)*



2b. Loads from the shotcrete of the primary support due to creep:



2b. Loads from the shotcrete of the primary support due to creep:

Analysis results for:

$$t_M / t_A = 50\text{cm} / 20\text{cm} = 2.5 \quad , \quad p_r / p_o = 0.30 \ (\lambda = 70\%)$$

Δt = time delay in the construction of the final lining

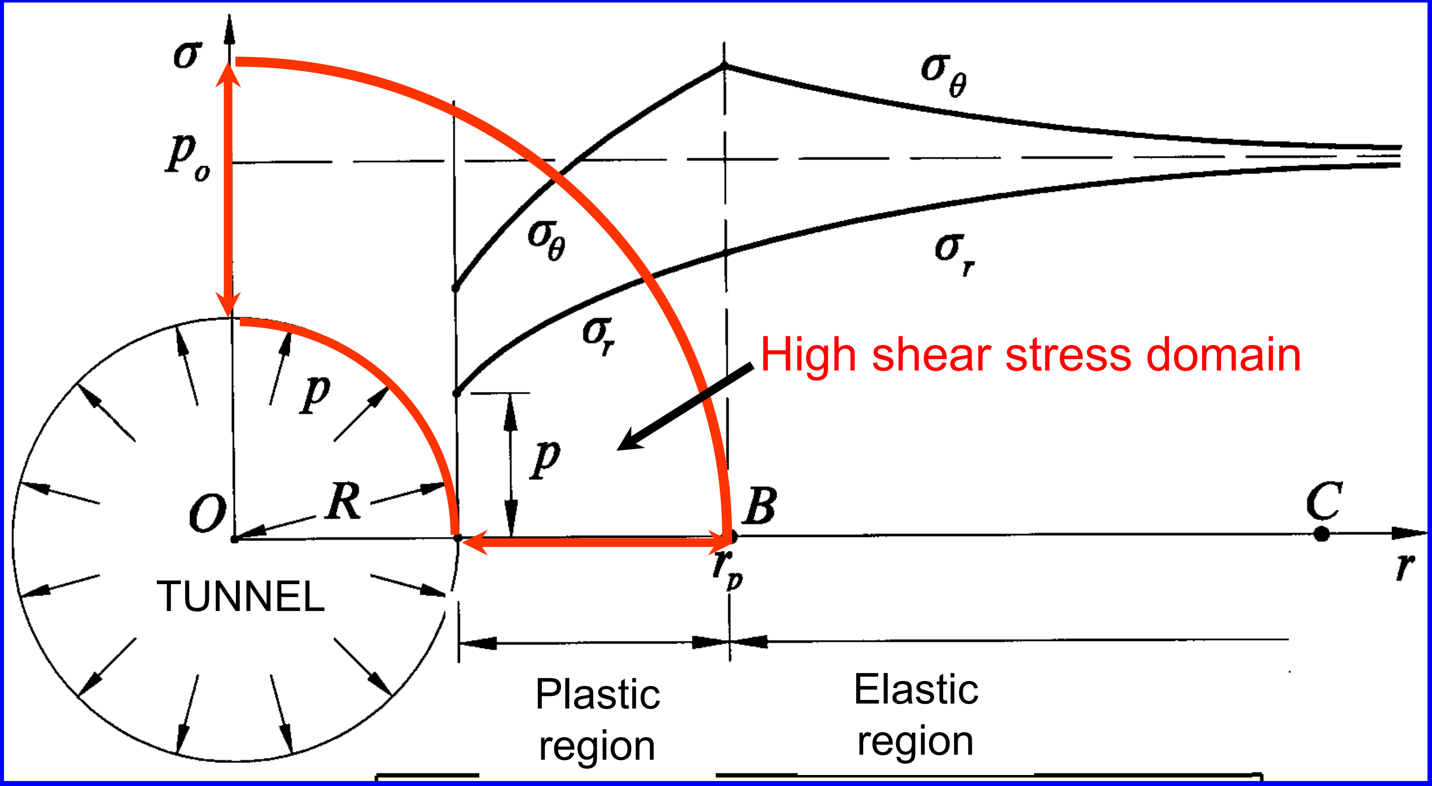
p = time dependent pressure on the final lining

Δt (months)	Equiv. pressure p (kPa)	p / p_o for a tunnel depth H=150m ($p_o = \gamma H$)
1	82	$p = 0.023\ p_o$
2	59	$p = 0.016\ p_o$
3	42	$p = 0.012\ p_o$
4	30	$p = 0.008\ p_o$
6	15	$p = 0.004\ p_o$
8	8	$p = 0.002\ p_o$
12	2	$p = 0.0005\ p_o$
24	0.4	$p = 0.000002\ p_o$

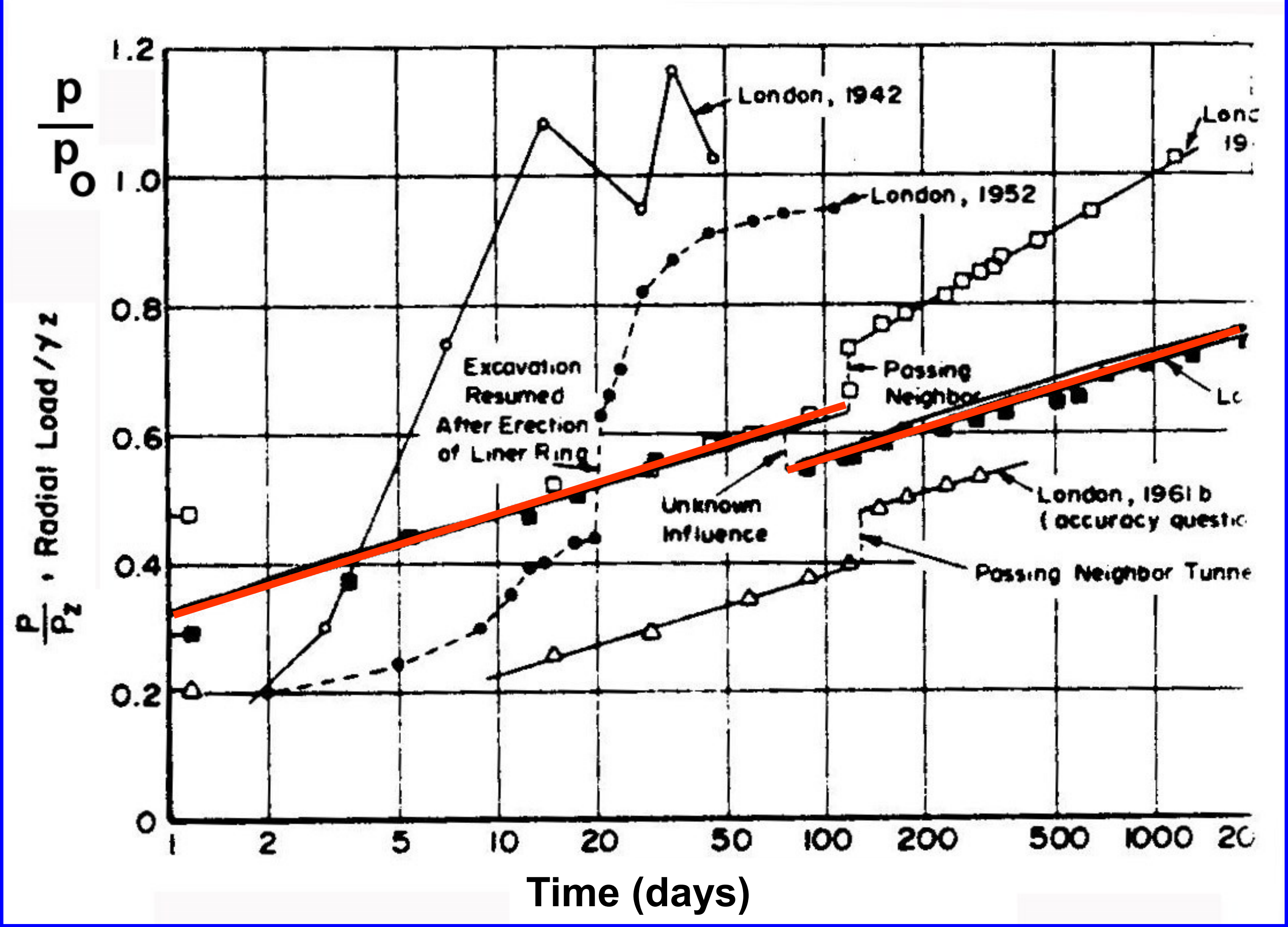
Origin of ground loads on the final lining of tunnels

3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground squeezing

- High shear stress $q = \sigma_\theta - \sigma_r$ caused by tunnel excavation can cause creep to some ground types (with large $N_s = 2\ p_o / \sigma_{cm}$)
- As the final lining prevents the development of creep strains, the creep load on the final lining increases with time



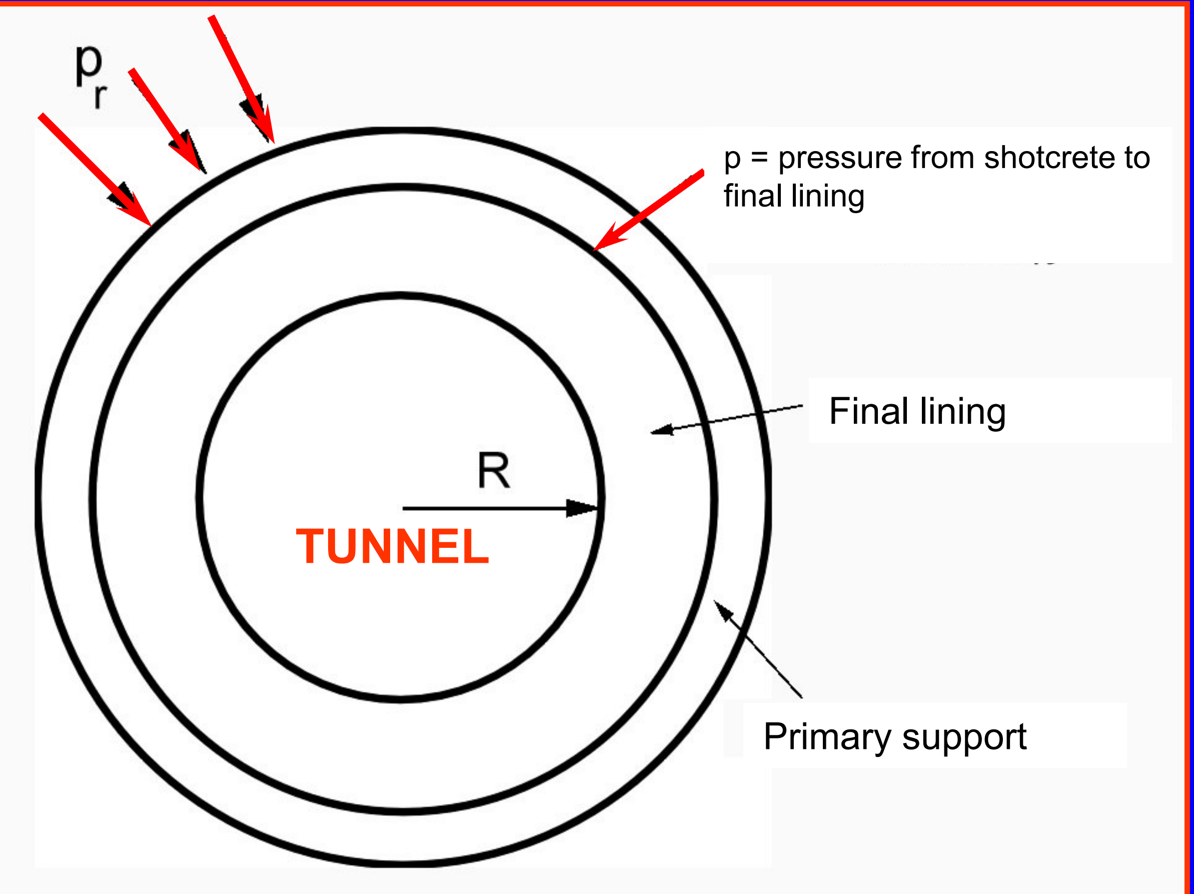
3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground squeezing



Evolution of ground pressure on the final lining in London Clay (Peck, 1969)
Very few such measurements exist in the literature

3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground squeezing

Result: Gradual loading of the final (and primary) lining, as the stiff lining prevents the gradual inward ground deformation due to creep. Ground pressure p_r on the lining increases with time

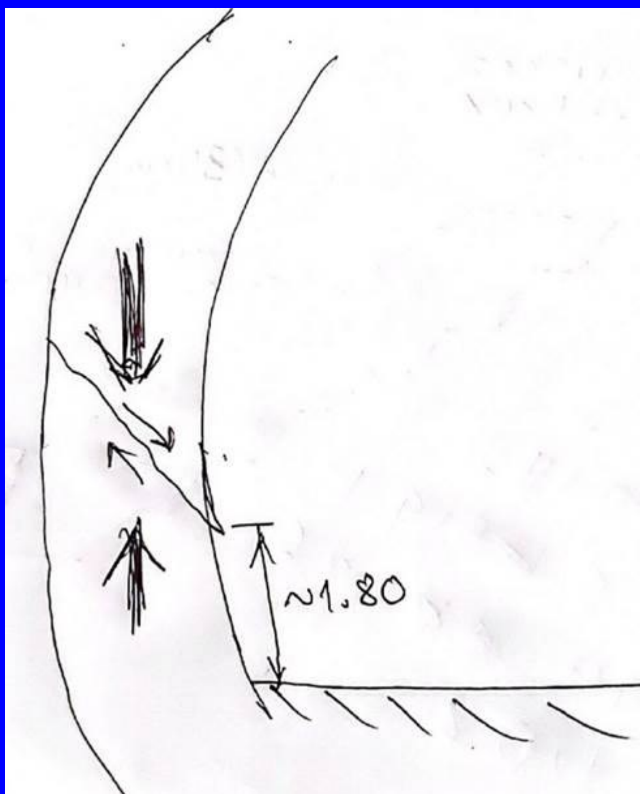


Indication of ongoing creep: Tendency for continued inward convergence of the primary support, far behind the excavation face, eventually causing cracking of the shotcrete.

In severe creep: Same behaviour on the final lining.

Creep is more pronounced in cases of very large values of $N_s = 2\ p_o\ /\ \sigma_{cm}$

Long-term compressive strength failure of the side-wall, due to large creep loads



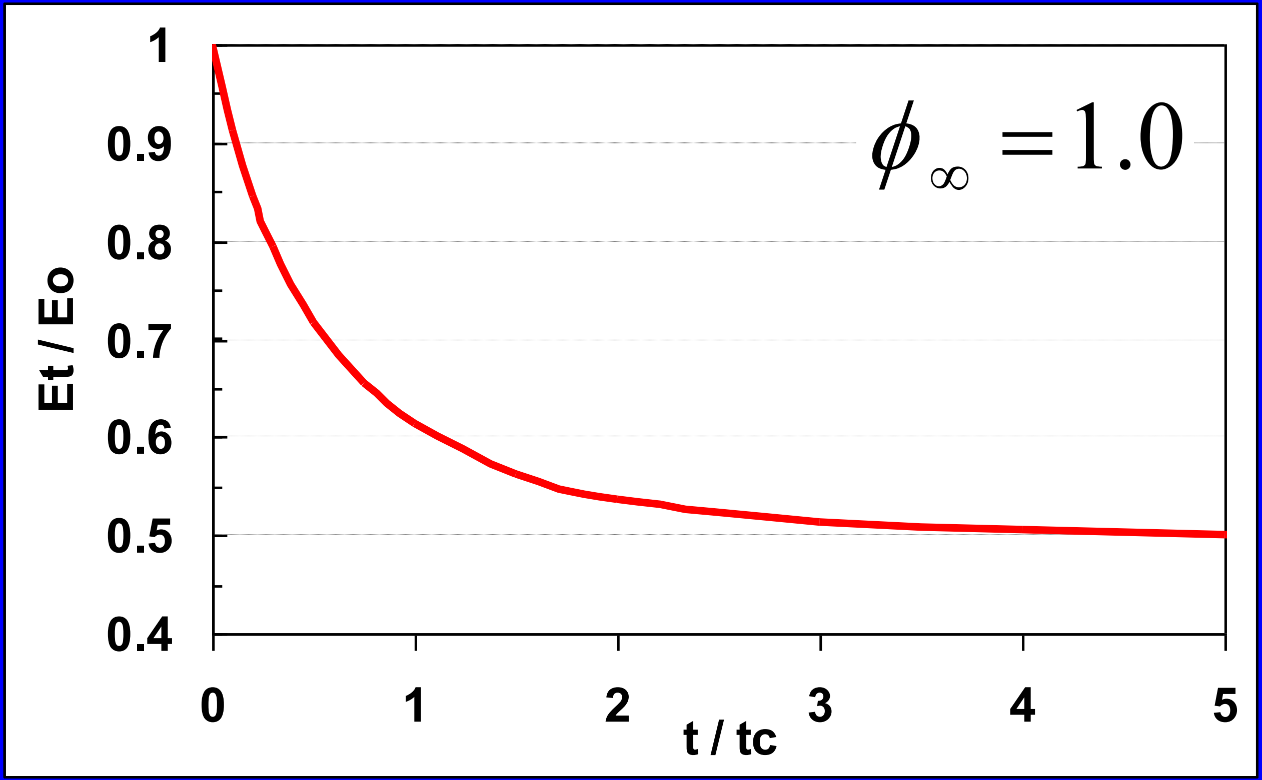
3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground squeezing

Ground creep model: $$E_t = \frac{E_o}{1+\phi_\infty\left[1-\exp\left(-t/t_c\right)\right]}$$

$$E_{t\infty} = \frac{E_o}{1+\phi_\infty}$$

$$\phi_\infty = \frac{\varepsilon_{t\infty}}{\varepsilon_e} = \frac{\text{final creep strain}}{\text{elastic strain}}$$

Shear-induced creep: K = ct, G- time dependent $\Rightarrow$ $$G_t = \frac{3KE_t}{9K-E_t}$$



3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground squeezing

Ground creep model:

Increasing squeezing ↓

Magnitude of ground creep	$\phi_\infty = \dfrac{\varepsilon_{t\infty}}{\varepsilon_e}$	$\dfrac{E_{t\infty}}{E_o} = \dfrac{1}{1+\phi_\infty}$
Negligible	0.05	0.95
Very small	0.1	0.91
Small	0.25	0.80
Medium	0.5	0.67
Large	1.0	0.50
Very large	2.5	0.29

ϕ_∞ = ratio of final creep strain to the elastic strain

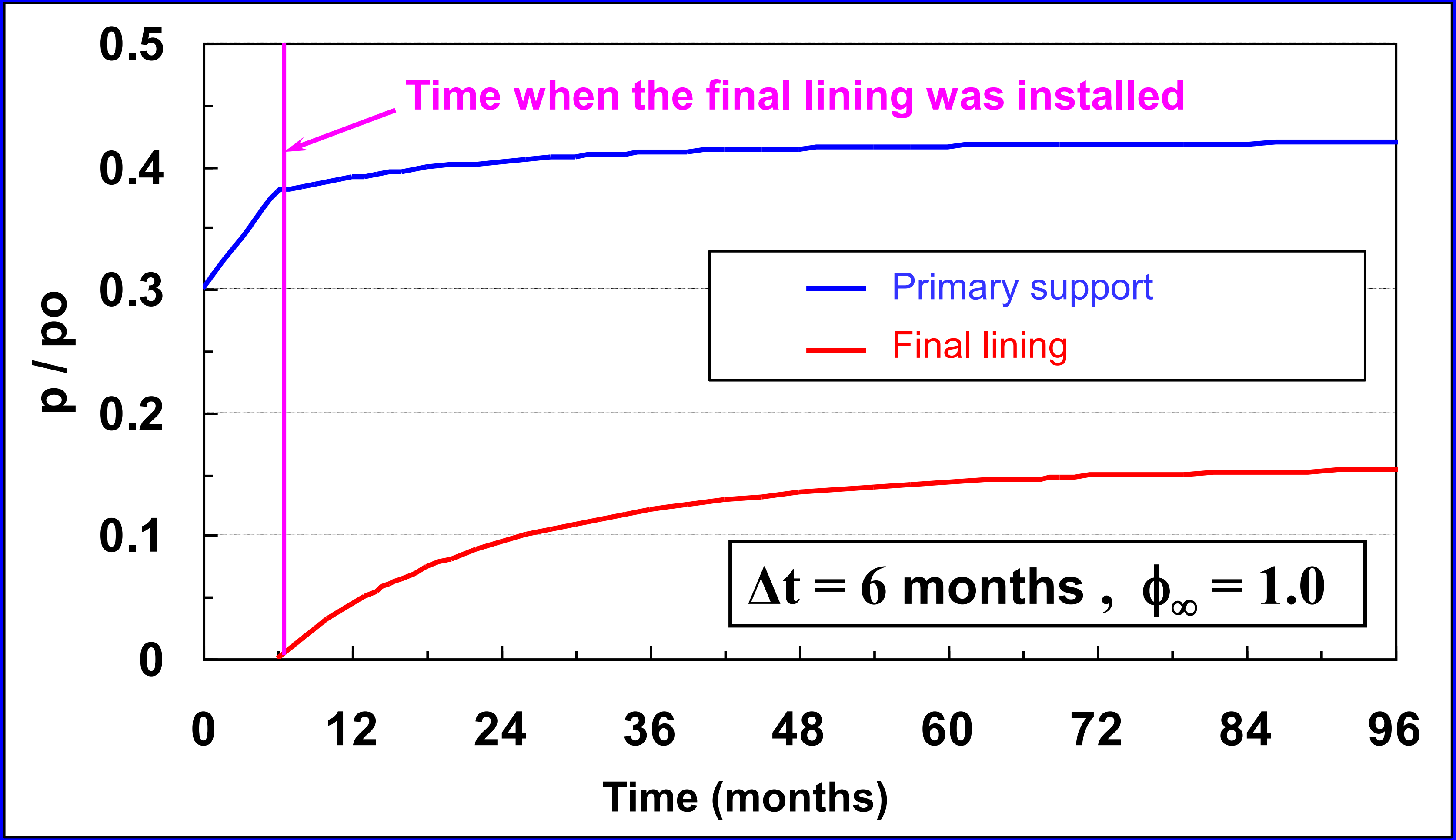
So, ϕ_∞ expresses the ratio of long-term wall convergence to the immediate (elastic) wall convergence

3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground squeezing

Results of creep analysis for:

$E_o = 400$ MPa , $\nu_o = 0.30$, $t_c = 24$ months , $\lambda = 0.70$, $B = 12$m

$t_M / t_A = 50\text{cm} / 20\text{cm} = 2.5$, no creep in concrete & shotcrete



3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground squeezing

Results of creep analysis for:

Values of p / p_o

Ground creep	ϕ_∞	Time delay in the construction of the final lining after excavation (Δt)		
		3 months	6 months	12 months
Negligible	0.05	0.027	0.023	0.018
Very small	0.1	0.050	0.044	0.034
Small	0.25	0.107	0.092	0.068
Medium	0.5	0.169	0.141	0.101
Large	1.0	0.233	0.186	0.125
Very large	2.5	0.274	0.198	0.138

p = *long-term pressure on the final lining*

p_o = *initial geostatic pressure*

3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground squeezing

Results of creep analysis for:

Values of $p\ (kPa)$ for $p_o = 3.6\ MPa$ (tunnel depth H=150m)

Ground creep	ϕ_∞	Time delay in the construction of the final lining after excavation (Δt)		
		3 months	6 months	12 months
Negligible	0.05	97	83	65
Very small	0.1	180	158	122
Small	0.25	385	331	245
Medium	0.5	608	508	364
Large	1.0	839	670	450
Very large	2.5	986	713	497

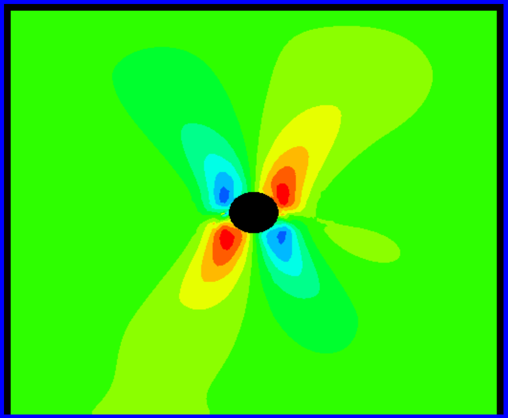
p = *long-term pressure on the final lining*

p_o = *initial geostatic pressure*

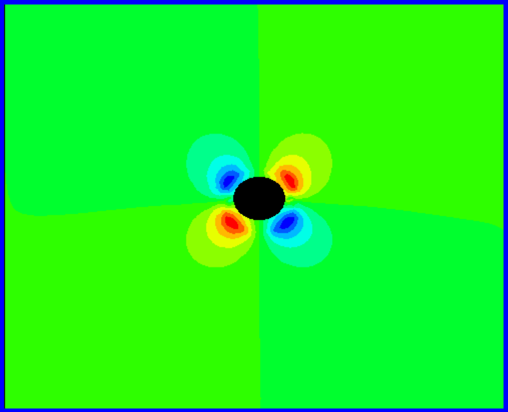
3. Loads due to ground creep, caused by the shear (mainly) stress changes due to tunnel construction - ground squeezing

Influence of the horizontal stress coefficient $K = \sigma_{ho} / \sigma_{vo}$:

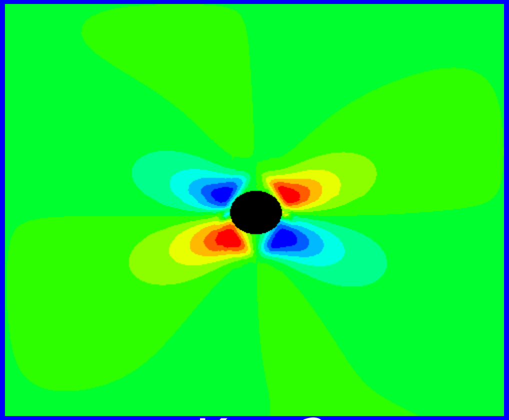
1. In the distribution of shear stresses:



K = 0.5

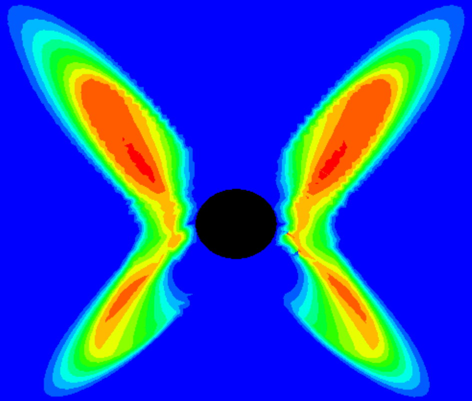


K = 1

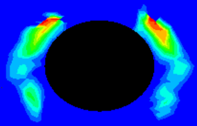


K = 2

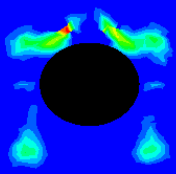
2. In the distribution of creep strains:



K = 0.5



K = 1

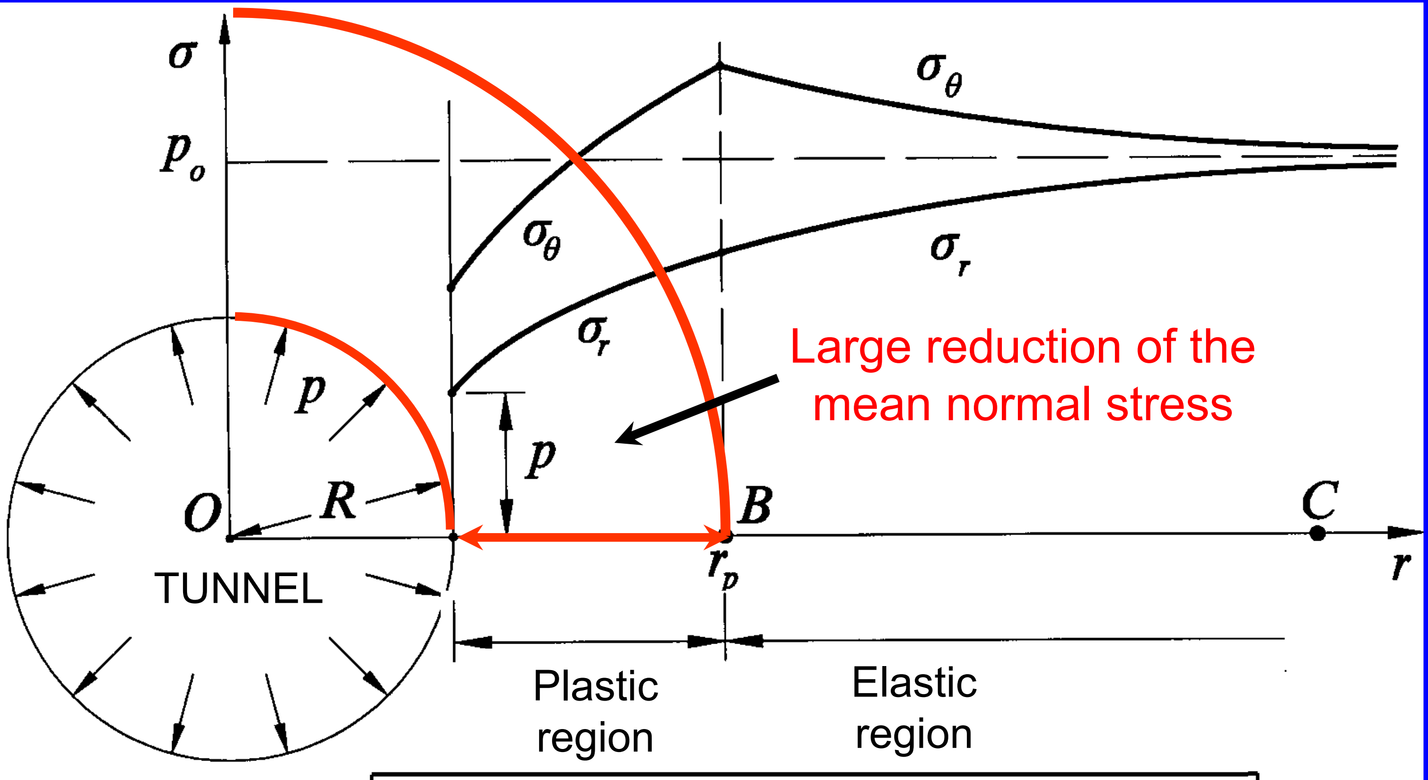


K = 2

Origin of ground loads on the final lining of tunnels

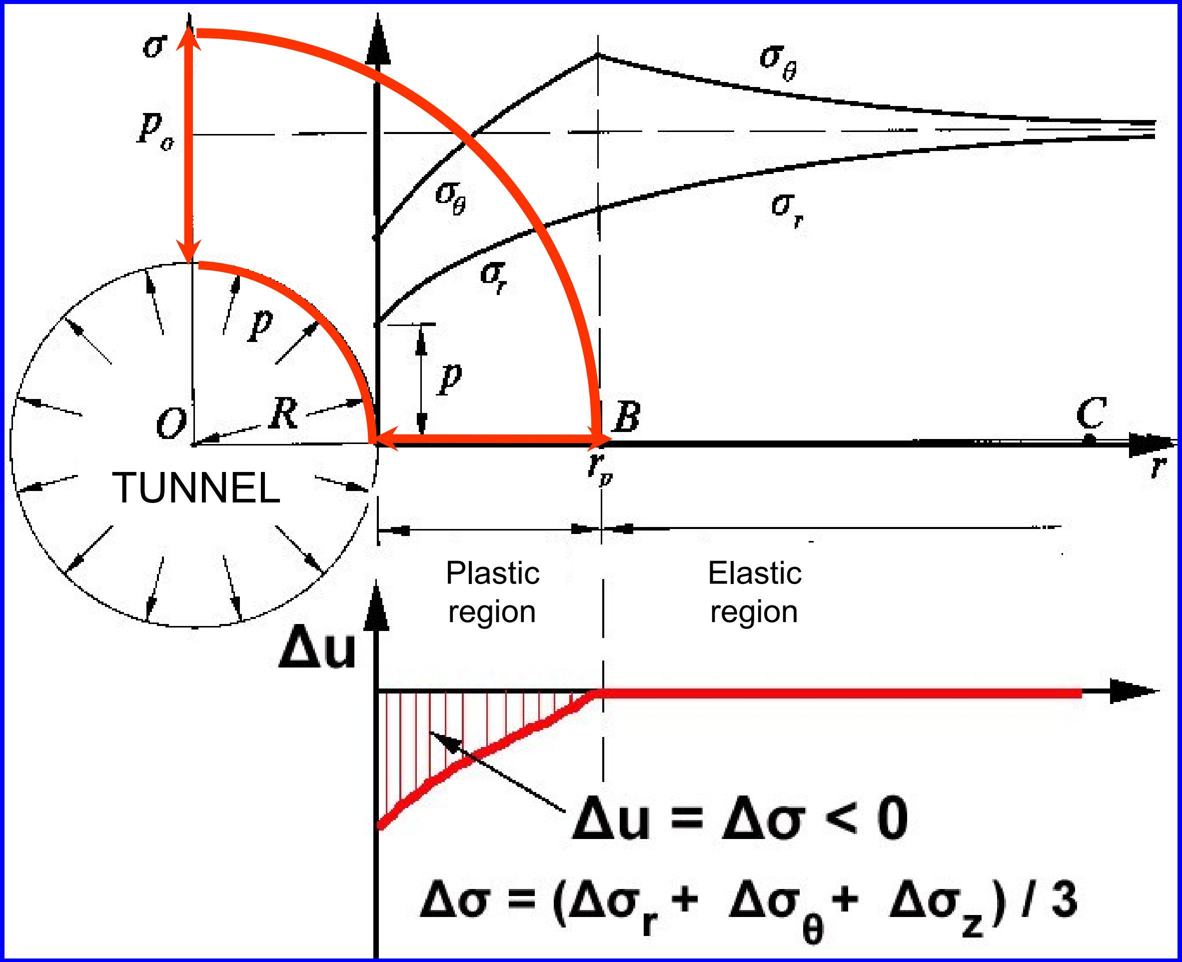
4. Loads due to ground swelling caused by negative ground consolidation

- Large reduction of the mean normal stress $\sigma = (\sigma_r+\sigma_\theta+\sigma_z) / 3$ in the plastic region → development of large negative excess pore pressures ($\Delta u < 0$)
- Gradual consolidation with suction of water and swelling
- If the tendency of ground swelling is obstructed by the presence of lining, large ground pressures are exerted on the lining (to increase the effective stress and thus reduce the tendency of swelling).



4. Loads on the lining due to ground swelling

Reduction of $\sigma = (\sigma_r+\sigma_\theta+\sigma_z) / 3$ in the plastic region, and thus development of $\Delta u < 0$



$$N_s = \frac{2\,\sigma'_o}{\sigma_{cm}}$$

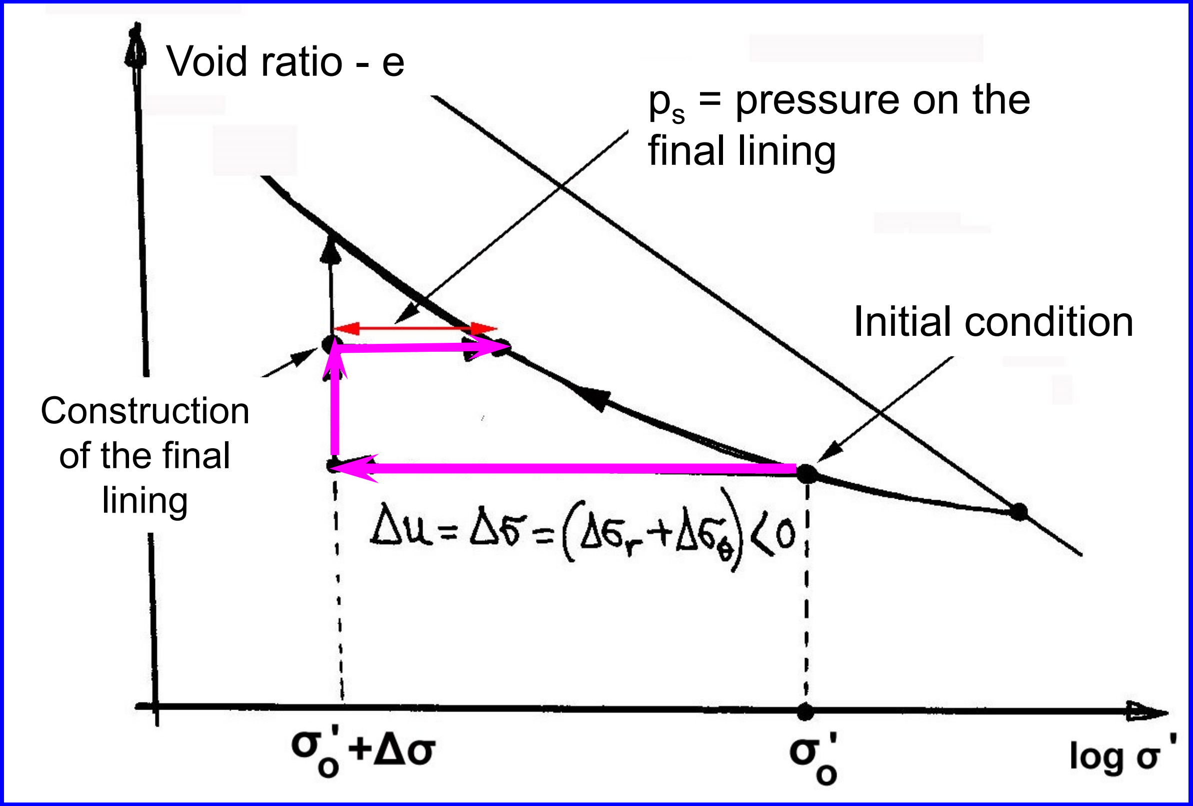
$$k = \frac{1+\sin\varphi}{1-\sin\varphi}$$

$$\lambda_{cr} = 1 - \frac{2}{1+k}\left(1-\frac{1}{N_s}\right)$$

$$\frac{\Delta u_{av}}{\sigma'_o} = \frac{\Delta\sigma_{av}}{\sigma'_o} = \frac{1}{2}\left(1-\frac{1}{N_s}\right)\left[\left(\frac{1-\lambda}{1-\lambda_{cr}}\right)-1\right]$$

4. Loads on the lining due to ground swelling

If the tendency of ground swelling is prevented, loads develop on the lining



Time delay in the construction of the final lining: Δt

Characteristic consolidation time (U=90%) : $t_c = \dfrac{r_p^2}{c}$

r_p = Radius of plastic zone

c = consolidation coefficient

4. Loads on the lining due to ground swelling

Results of ground swelling analysis:

Values of p / p_o

Time delay in construction of the final lining	Coefficient of consolidation c (m^2 / έτος)			
	Medium clay 10 m^2 / έτος	Stiff clay 25 m^2 / έτος	Hard clay 50 m^2 / έτος	Weak rock 100 m^2 / έτος
3 months	0.197	0.177	0.148	0.104
6 months	0.184	0.148	0.104	0.052
1 year	0.159	0.104	0.052	0.014
1.5 years	0.138	0.074	0.027	0.004
2 years	0.120	0.052	0.014	0.0001
3 years	0.091	0.027	0.004	≈ 0

p = *long-term pressure on the final lining*

p_o = *initial geostatic pressure*

4. Loads on the lining due to ground swelling

Results of ground swelling analysis:

Values of p (kPa) for $p_o = 3.6\ MPa$ (tunnel depth H=150m)

Time delay in construction of the final lining	Coefficient of consolidation c (m^2 / έτος)			
	Medium clay 10 m^2 / έτος	Stiff clay 25 m^2 / έτος	Hard clay 50 m^2 / έτος	Weak rock 100 m^2 / έτος
3 months	709	637	533	374
6 months	662	533	374	187
1 year	572	374	187	50
1.5 years	497	266	97	14
2 years	432	187	50	6
3 years	328	97	14	≈ 0

p = *long-term pressure on the final lining*

p_o = *initial geostatic pressure*

Loading of the final lining of tunnels - Summary

	Origin of loading	Range of values: p (kPa)	$H_T = p / \gamma$ (m)	p / p_o (%)
1	Rock bolts	20 ÷160	1 ÷ 8	0.6 ÷ 4.5 *
	Steel sets	40 ÷110	2 ÷ 5.5	1.2 ÷ 3 *
	Shotcrete (due to SF)	40 ÷100	2 ÷ 5	1.2 ÷ 2.8 *
	Shotcrete (due to creep)	0 ÷ 80	0 ÷ 4	0 ÷ 2.3 *
2	Ground creep (squeezing)	0 ÷1000 *	0 ÷ 50	0 ÷ 30
	Ground swelling	0 ÷ 700 *	0 ÷ 35	0 ÷ 20

* *For a tunnel depth H = 150 m.* $p_o = \gamma H = 3.6$ *MPa*

Sum of 1 : p = 100 ÷ 450 kPa , H_T = 5 ÷ 22 m = (0.5 ÷ 2) B

Sum of 2 : p = 0 ÷ 1700 kPa, H_T = 0 ÷ 85 m = (0 ÷ 8) B

Loads on the final lining of tunnels - Modelling

1. 1-D models (beams on Winkler springs)

   Application of known ground pressures (p)

2. 2-D models (e.g. finite elements)

Origin of loading		Modelling
1	Rock bolts and steel sets	Complete deactivation
1	Shotcrete (due to SF)	Reduction of the E-modulus of shotcrete to cause the required pressure (p) on the final lining NOTE: Full deactivation of shotcrete is probably almost equivalent because the larger effect due to E=0 is compensated by stress redistribution in the surrounding ground
1	Shotcrete (due to creep)	
2	Ground creep - squeezing	Ground creep model, or suitable reduction of the ground E-modulus in the plastic zone NOTE: Difficult to estimate the suitable E-modulus reduction corresponding to specific creep intensity
2	Ground swelling	Ground consolidation model, or suitable reduction of the ground E-modulus in the plastic zone NOTE: Difficult to estimate the suitable E-modulus reduction corresponding to specific swelling intensity

Conclusions

1. 1-D analysis with beam models and Winkler springs:

- Requires pre-defined ground loads. The values of these loads involve appreciable uncertainty because they neglect ground-lining interaction, ground squeezing and swelling.
- Can be used with caution in relatively simple loading cases (no squeezing, no swelling)

2. 2-D analysis with finite elements:

- In relatively simple cases (no squeezing, no swelling), full deactivation of all temporary support measures gives reasonable results.
- In cases with squeezing / swelling, a suitable E-modulus reduction in a ground zone around the tunnel (e.g. plastic zone) can give acceptable results (difficulty: estimate modulus reduction to model ground creep)