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 5: Tunnel face stability

31.07.2023

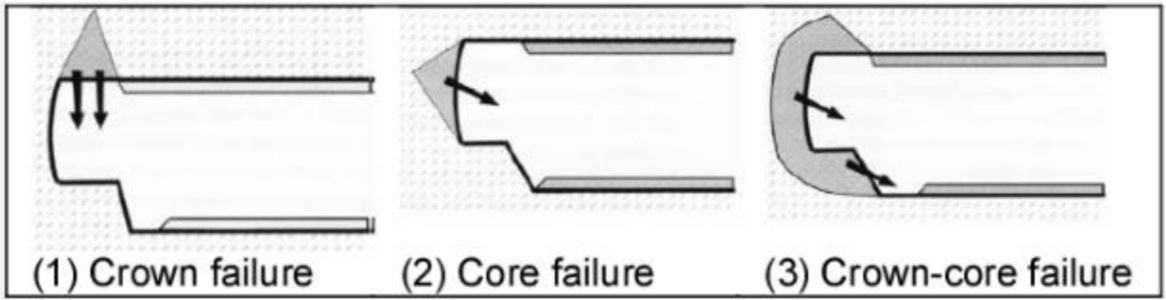
Tunnel Face Stability

Old-fashioned tunnel face support with boarding and fore-poling using steel (rail tracks) and wooden poles (pass-avant). Note the sliding joints on the steel ribs to accommodate larger wall convergence in squeezing ground

Photo: Deep copper mine in Chile.

Tunnel Face Stability

Objective 1: Prevent excavation face instability


(1) Crown failure
(2) Core failure
(3) Crown-core failure

Face instability in weakly cemented neogene deposits

Patras by-pass tunnels (1997)

Face instability in a mining tunnel

Partial face instability of the Othrys railway tunnel (2020)

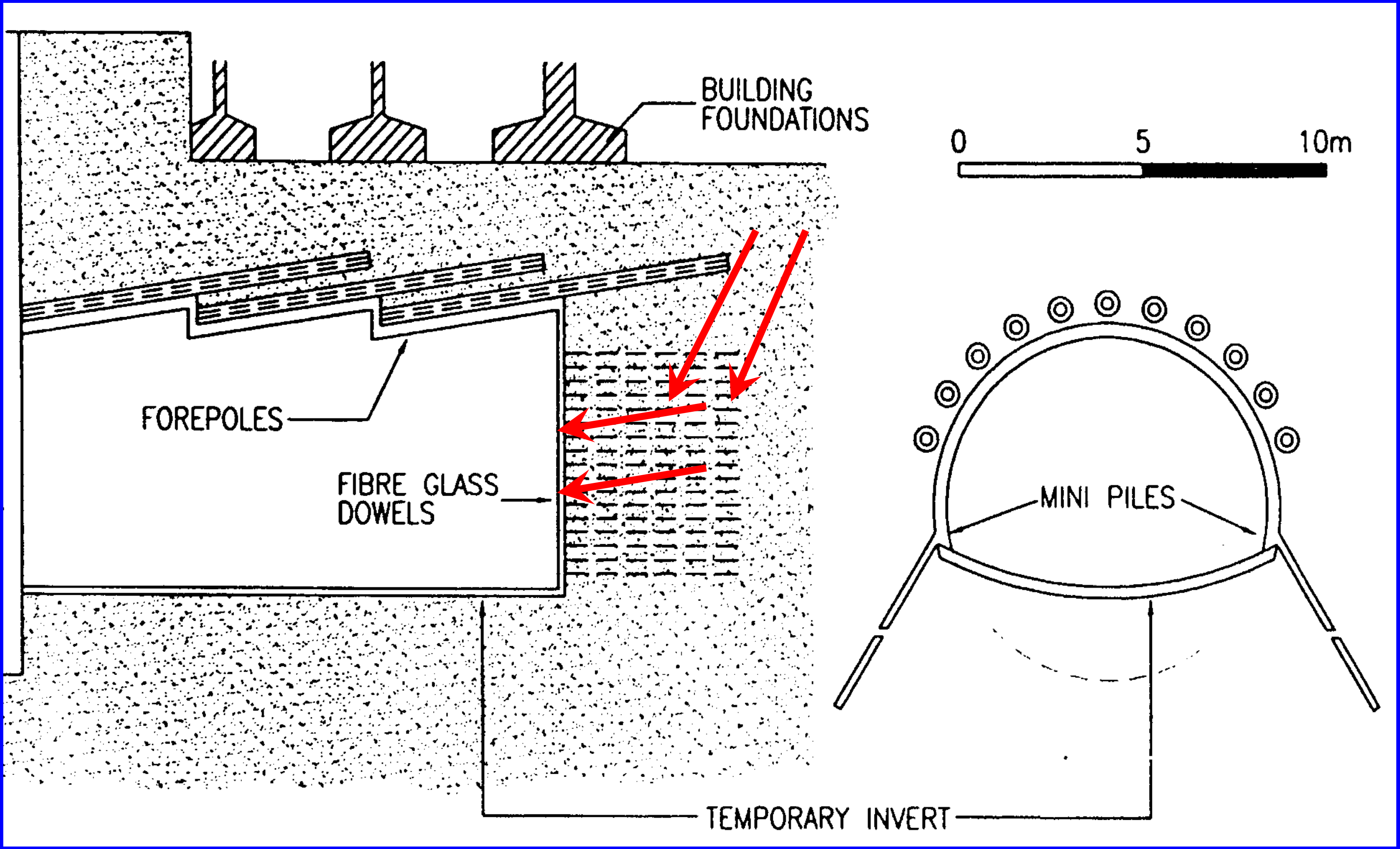
(most probably due to increased water pressures behind the shotcrete cover – placed during a prolonged interruption of tunnel advance)

Stable and unstable tunnel face in a thickly bedded sandstone. Face becomes unstable in fractured zones, due to lack of cohesion between blocks (open fractures).

Video of face instability in a heavily fractured gneiss

Tunnel Face Stability

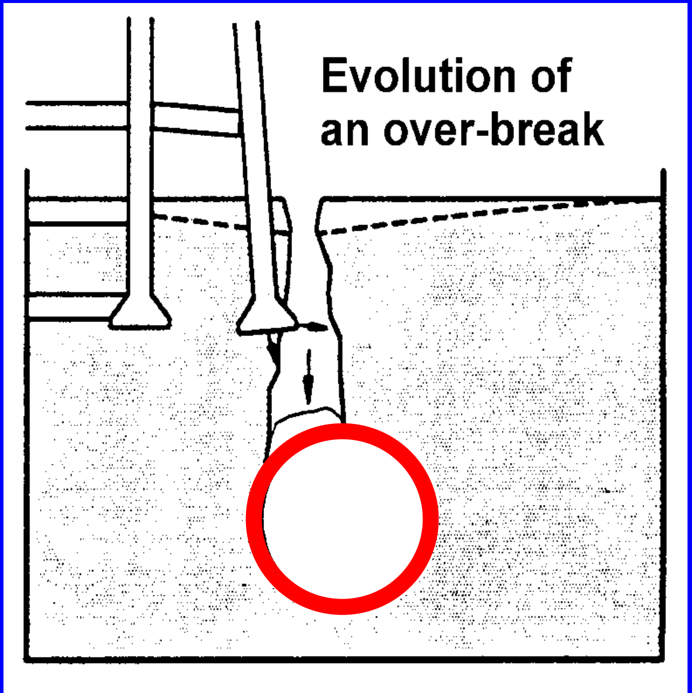
Objective 2: Reduce “face-take” (inward face movement) to reduce ground surface settlement in shallow (usually urban) tunnels



Tunnel Face Stability

Objective:

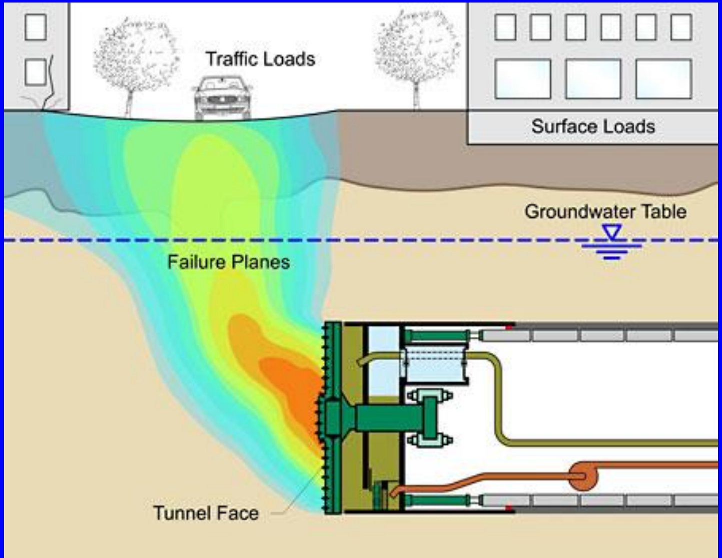
2. Reduce “face-take” (inward face movement) to reduce ground surface settlement in shallow (usually urban) tunnels



Athens Metro, Karaiskaki square – TBM face collapse reaching ground surface (1997)

Tunnel Face Stability

Athens Metro, Panepistimiou Av.
Catastrophic TBM face collapse (1997)

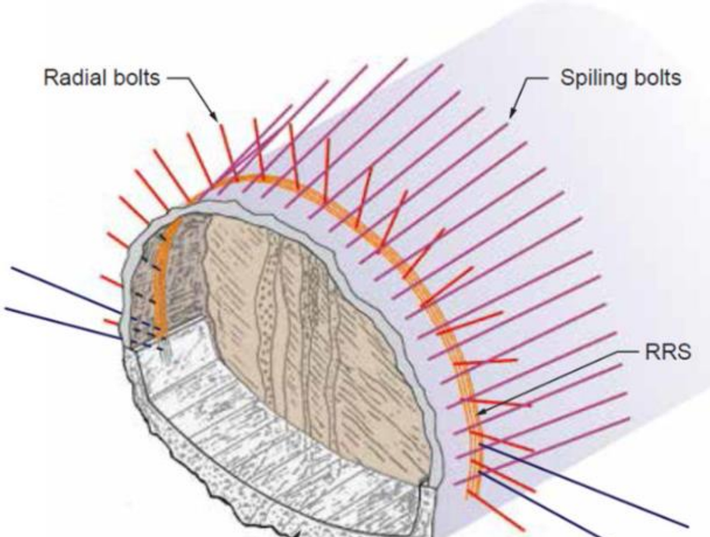


Tunnel Face Stability

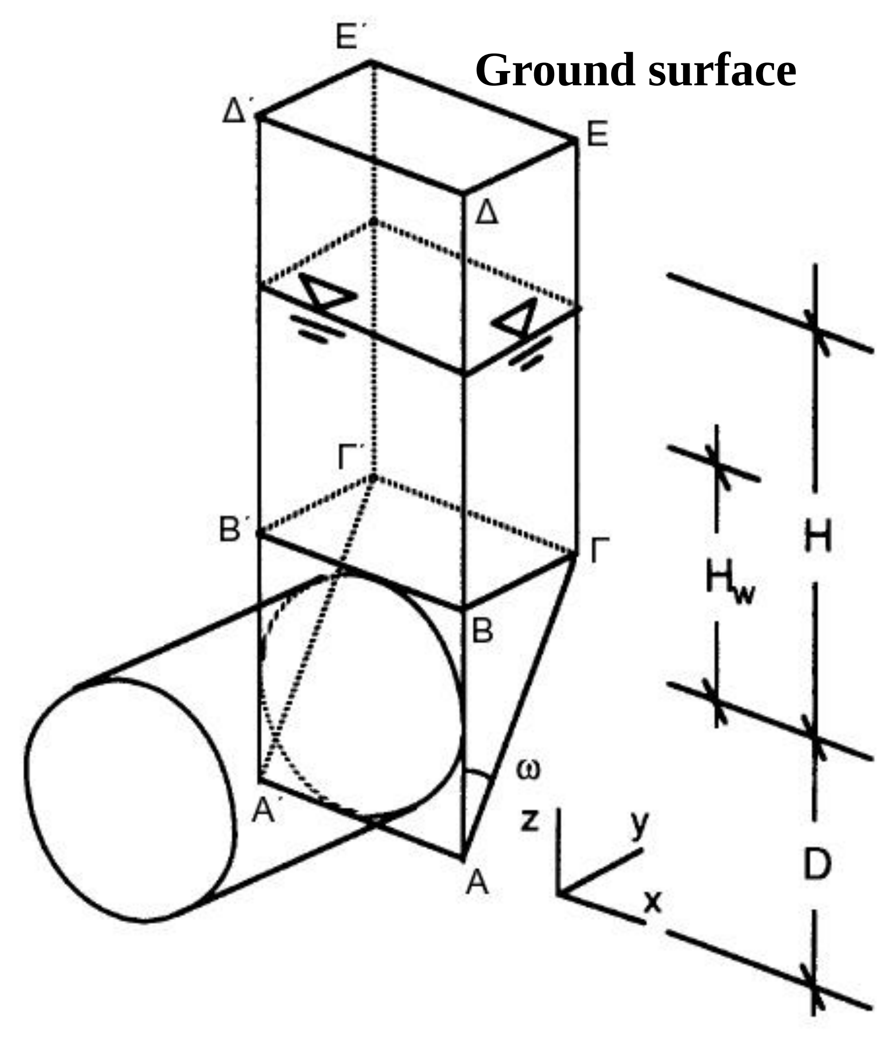
Athens Metro, Douk. Plakentias Av. Catastrophic face collapse during conventional tunnelling (2003)

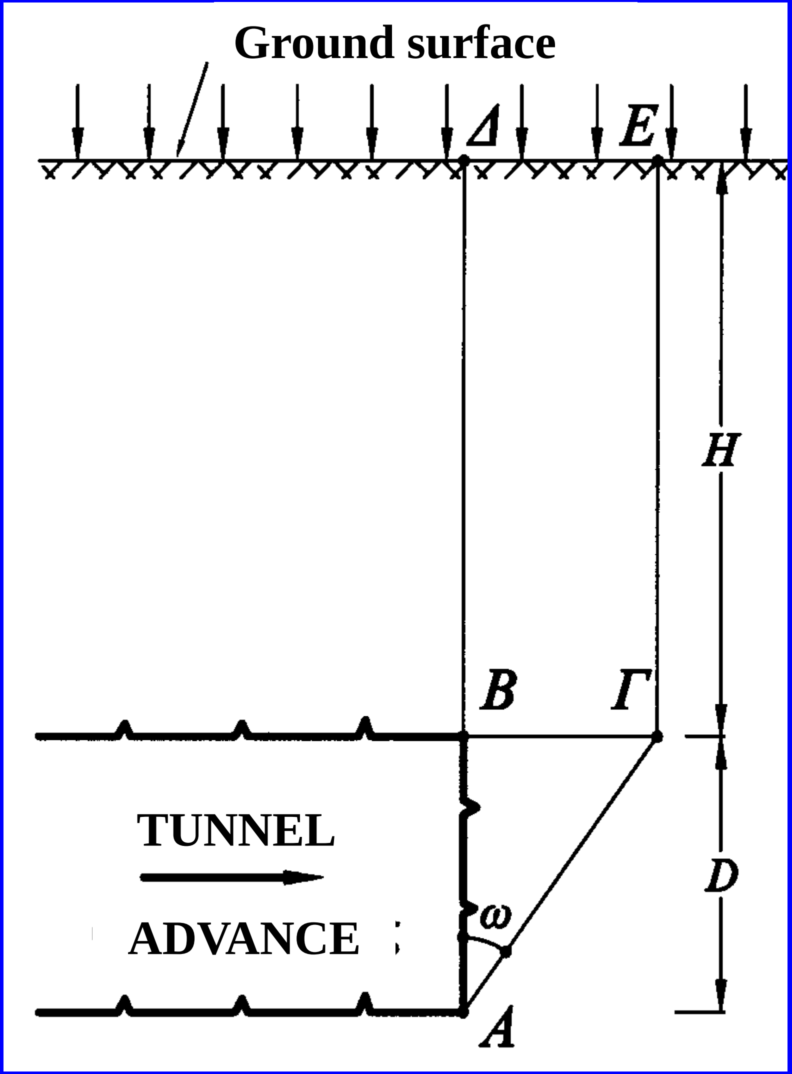
Tunnel Face Stability

Objective 3: Avoid crest raveling (gradual roof collapse in low cohesion ground due to loss of stability of particles, causing instability of adjacent particles) before placing temporary support (shotcrete)



Analysis of Tunnel Face Stability: Kovari-Anagnostou method



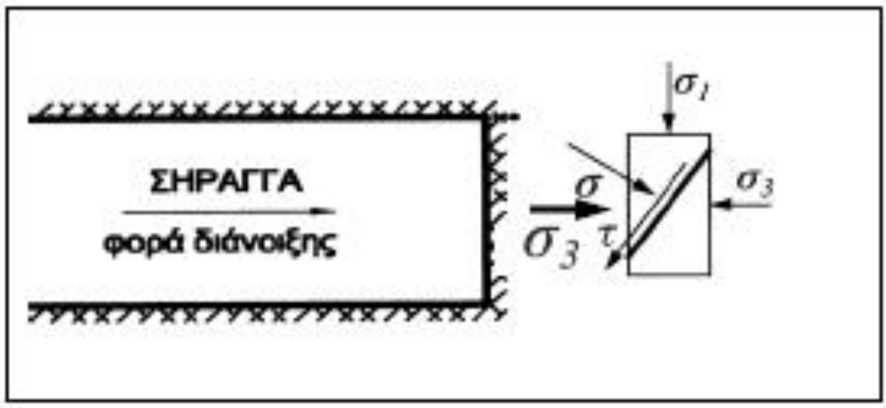


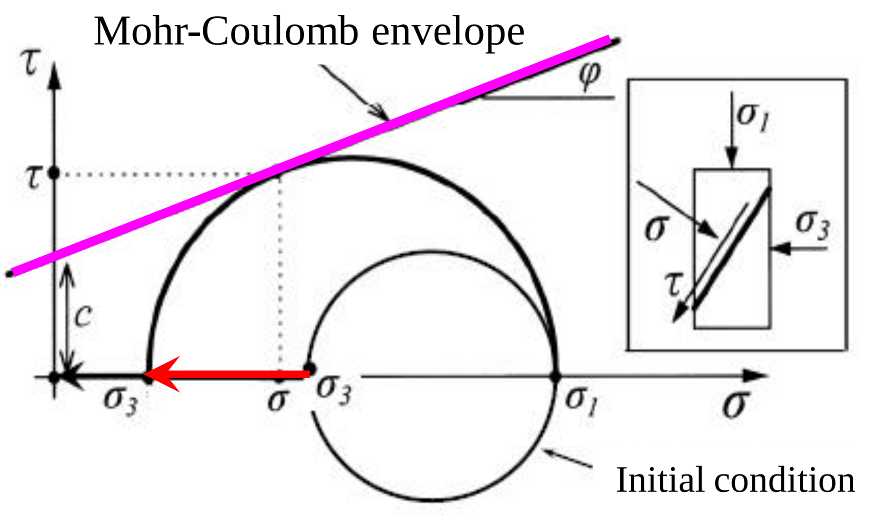
Analysis of Tunnel Face Stability: Kovari-Anagnostou method

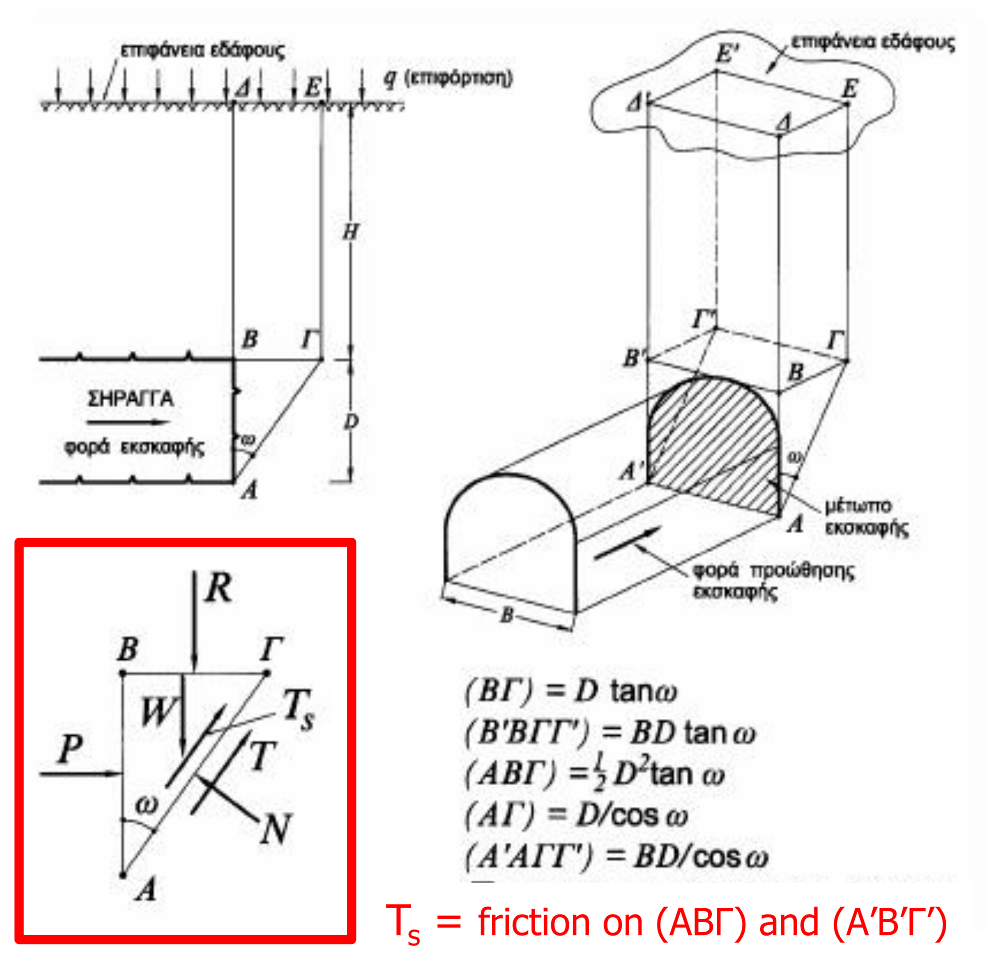
Tunnel face becomes unstable when the horizontal stress (σ_3) is reduced to a low value that causes failure (satisfies the failure criterion)

2D failure mechanism

Wedge type 3D failure mechanism (actions on the wedge)







$(B\Gamma) = D\tan\omega$

$(B'B\Gamma\Gamma') = BD\tan\omega$

$(AB\Gamma) = \frac{1}{2}D^2\tan\omega$

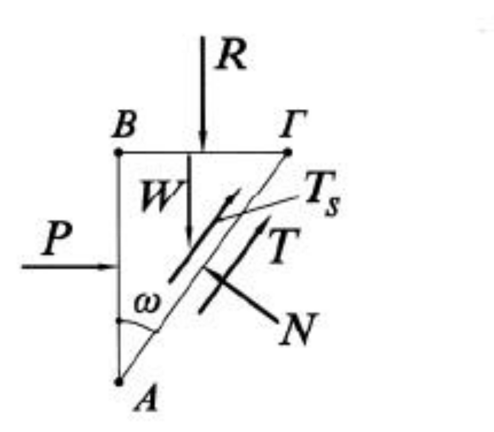
$(A\Gamma) = D/\cos\omega$

$(A'A\Gamma\Gamma') = BD/\cos\omega$

T_s = friction on (ΑΒΓ) and (Α′Β′Γ′)

Analysis of Tunnel Face Stability: Kovari-Anagnostou method

Force equilibrium of the sliding wedge:



- Force equilibrium along the sliding direction AΓ:

$$T + 2T_s = (R + W)\cos\omega - P\sin\omega$$

- Force equilibrium normal to the sliding direction AΓ:

$$N = (R + W)\sin\omega + P\cos\omega$$

- Shear force at sliding (*T*) satisfies the Morh-Coulomb criterion:

$$T = N\tan\phi + c\,(AA'\Gamma\Gamma') = N\tan\phi + c\frac{BD}{\cos\omega}$$

- Elimination of (*N*) and (*T*) gives the required **limiting support force (*P*)** on the tunnel face:

$$\boxed{P = \frac{R + W}{\tan(\omega + \phi)} - \frac{T_s + c\dfrac{BD}{\cos\omega}}{\cos\omega(\tan\omega + \tan\phi)}}$$

Calculation of parameters:

- Weight of the sliding wedge: $W = \gamma\frac{1}{2}BD^2\tan\omega$
- Vertical force on top of the wedge (σ_v = vertical pressure on wedge):

$$R = (B\Gamma B'\Gamma')\,\sigma_v = B\,D\tan\omega\,\sigma_v$$

Analysis of Tunnel Face Stability: Kovari-Anagnostou method

The vertical pressure σ_v on the top of the wedge is calculated from silo theory:

$$\sigma_v = \frac{L\gamma - c}{\lambda \tan\varphi}\left(1 - e^{-\lambda \tan\varphi \frac{H}{L}}\right)$$

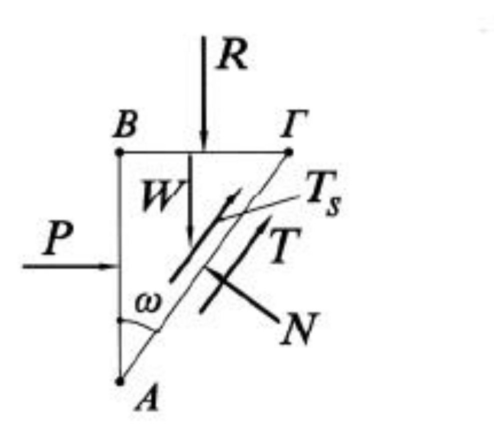
where:

H = tunnel depth (up to crest)

λ = coefficient of horizontal stress (silo effect), equal to about 1

and:

$$L = \frac{BD\tan\omega}{2\left(B + D\tan\omega\right)}$$



The friction (T_s) on the lateral triangles (ΑΒΓ) and (Α΄Β΄Γ΄) is calculated from silo theory and Mohr-Coulomb:

$$T_s = D^2 \tan\omega \left(c + \lambda_k \tan\varphi \frac{2\sigma_v + D\gamma}{3}\right)$$

Example - Short term face stability (φ=0, c=Su):

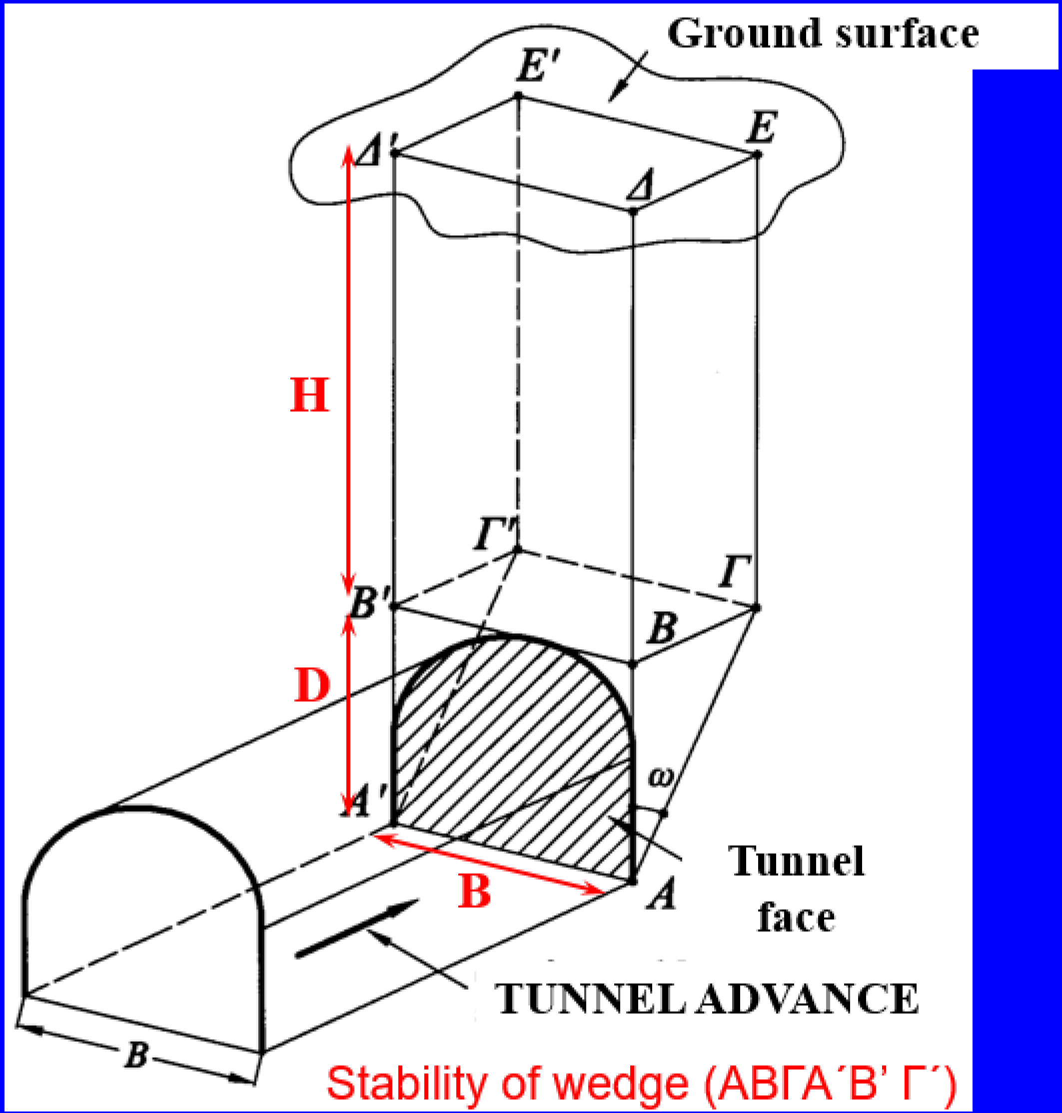
Required horizontal force (P) for limiting face stability:

$$P = BD\sigma_v + \frac{1}{2}\gamma BD^2 - s_u 2D \frac{D\sin\omega + B}{\sin 2\omega}$$

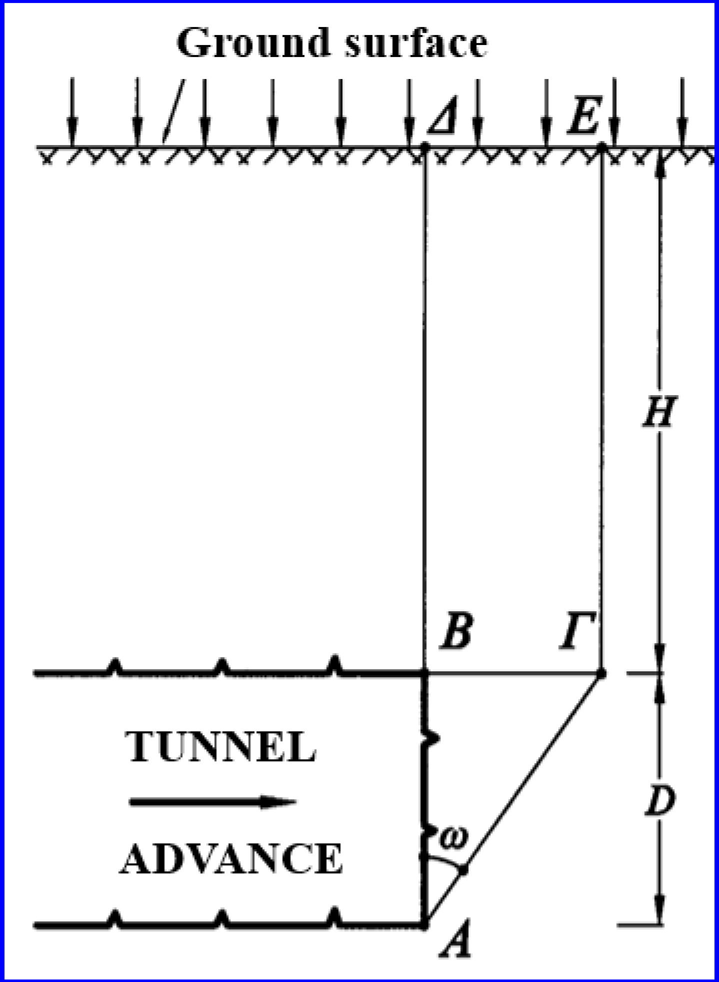
where: $\sigma_v = \gamma H\left(1 - \frac{S_u}{\gamma L}\right)$

$\sigma_v = 0$ if $S_u > \gamma L$

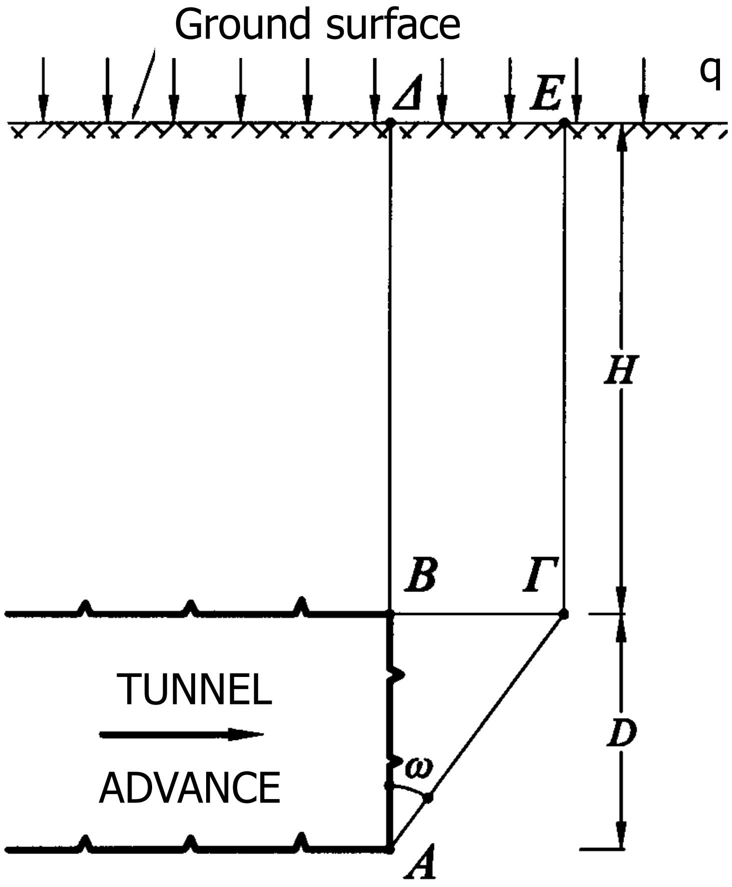
Analysis of Tunnel Face Stability
Simplified Kovari-Anagnostou method



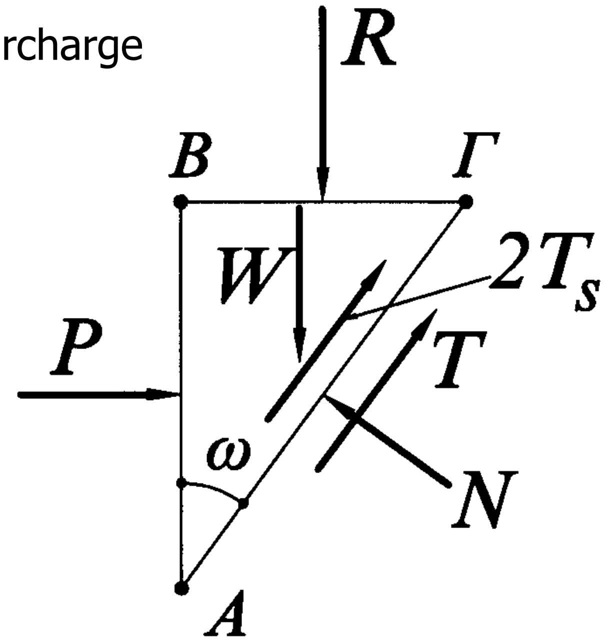
Stability of wedge (ΑΒΓΑ΄Β' Γ΄)



Analysis of Tunnel Face Stability
Simplified Kovari-Anagnostou method



q = surface surcharge



$(AB\Gamma) = \frac{1}{2} D^2 \tan\omega$

$(A\Gamma) = D/\cos\omega$

$(A'A\Gamma\Gamma') = BD/\cos\omega$

$(B\Gamma) = D\ \tan\omega$

$(B'B\Gamma\Gamma') = BD \tan\omega$

T_s = friction on (ABΓ) and (A′B′Γ′)

Analysis of Tunnel Face Stability
Simplified Kovari-Anagnostou method

Force equilibrium on wedge (ΑΒΓΑ'Β'Γ') in vertical and horizontal direction:

$$N = (R + W)\sin\omega + P\cos\omega$$

$$T + 2T_s = (R + W)\cos\omega - P\sin\omega$$

Weight: $W = \gamma\, B\,(AB\Gamma) = \frac{1}{2}\gamma\, D^2\, B\tan\omega$

Force on top of wedge: $R = \sigma_v (B'B\Gamma\Gamma') = \sigma_v BD\tan\omega$

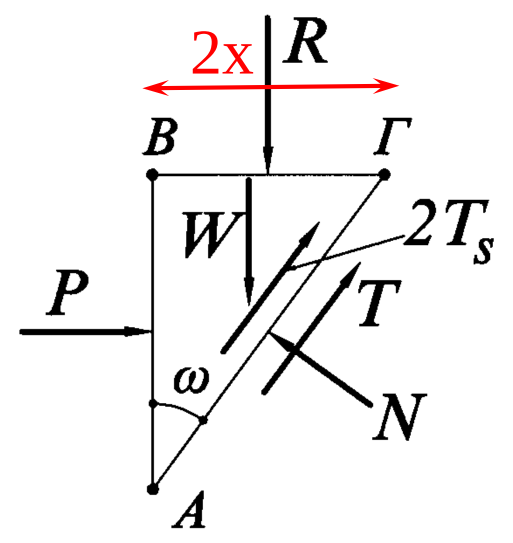
$\sigma_v = (1-\lambda)\, p_o$ = vertical stress at distance (x) ahead of tunnel face, using the deconfinement factor (λ) instead of the "Terzaghi silo theory"

F = safety factor = T_u / T

Side friction: $T_s = (AB\Gamma)\frac{\tau_f}{F} = \frac{1}{2}(D^2\tan\omega)\frac{\tau_f}{F}$

M-C limiting friction: $\tau_f = c + K\sigma_{vo}\tan\varphi$

Base friction: $T = \frac{1}{F}\left[c(\mathrm{A'A\Gamma\Gamma'}) + N\tan\varphi\right]$



Analysis of Tunnel Face Stability
Simplified Kovari-Anagnostou method

Combination of above gives the safety factor of face stability:

$$\boxed{F = \frac{N \tan\varphi + c\left(\mathrm{A'A\Gamma\Gamma'}\right) + 2\tau_f\left(\mathrm{AB\Gamma}\right)}{\left(R+W\right)\cos\omega - P\sin\omega}}$$

where:

$$N = \left(R+W\right)\sin\omega + P\cos\omega$$

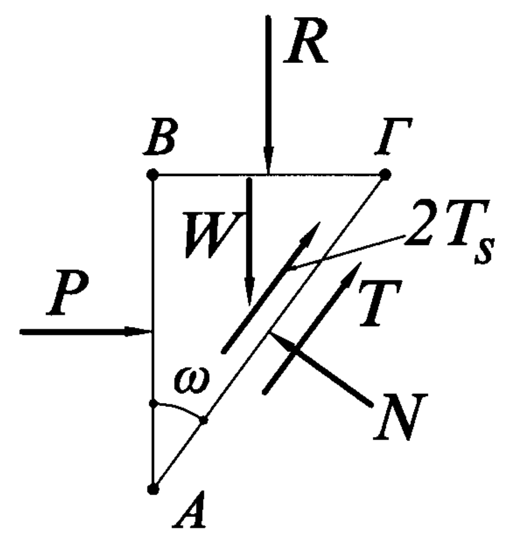
$$W = \frac{1}{2}\gamma\, D^2\, B\tan\omega$$

$$\omega \approx 45 - \frac{\varphi}{2}$$

$$R = \sigma_{vo}\, B\, D\, \tan\omega$$

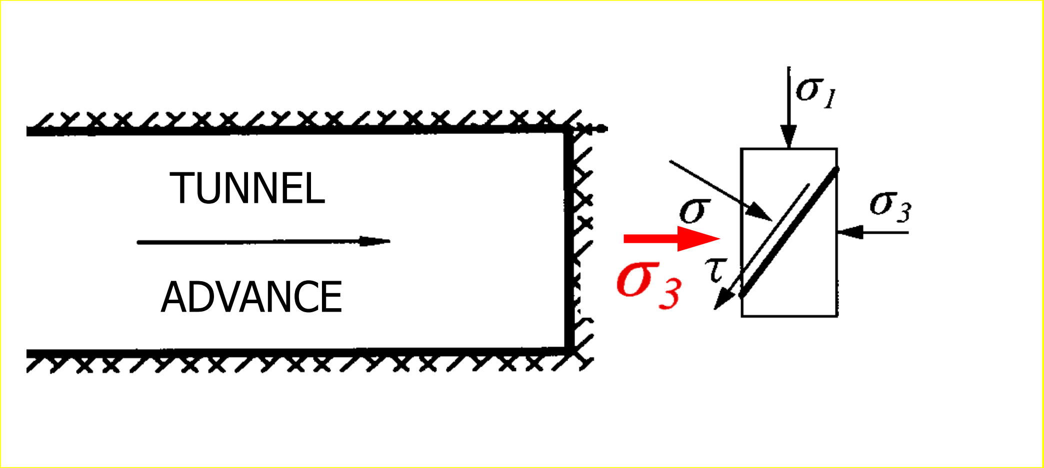
$$P = \sigma_3(ABB'A') = \sigma_3\, BD$$

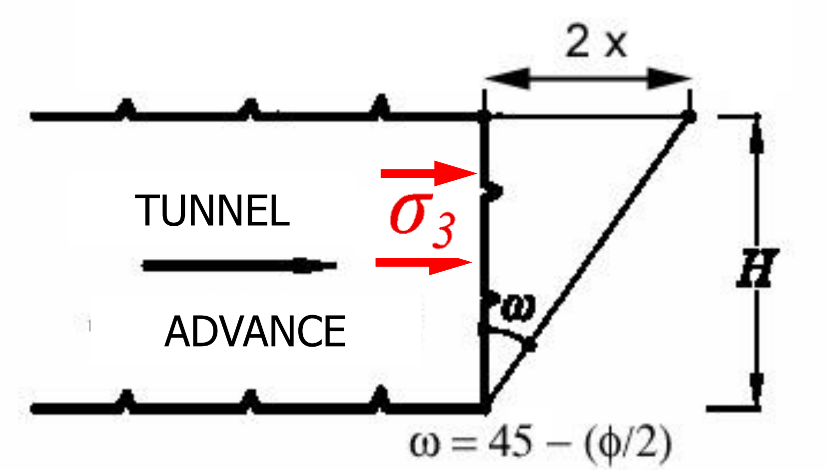
$$\tau_f = c + K\sigma_{vo}\tan\varphi$$



Analysis of Tunnel Face Stability

As the tunnel face advances, the horizontal stress at a specific location ahead of the face gradually reduces to zero ($\sigma_3 \Rightarrow 0$), possibly causing failure of the ground under uniaxial stress (σ_1)





The vertical stress (σ_1) also reduces due to (λ): $\sigma_1 = (1-\lambda)\, p_o$

Deconfinement factor (λ) depends on the distance (x) of the middle of the wedge from tunnel face:

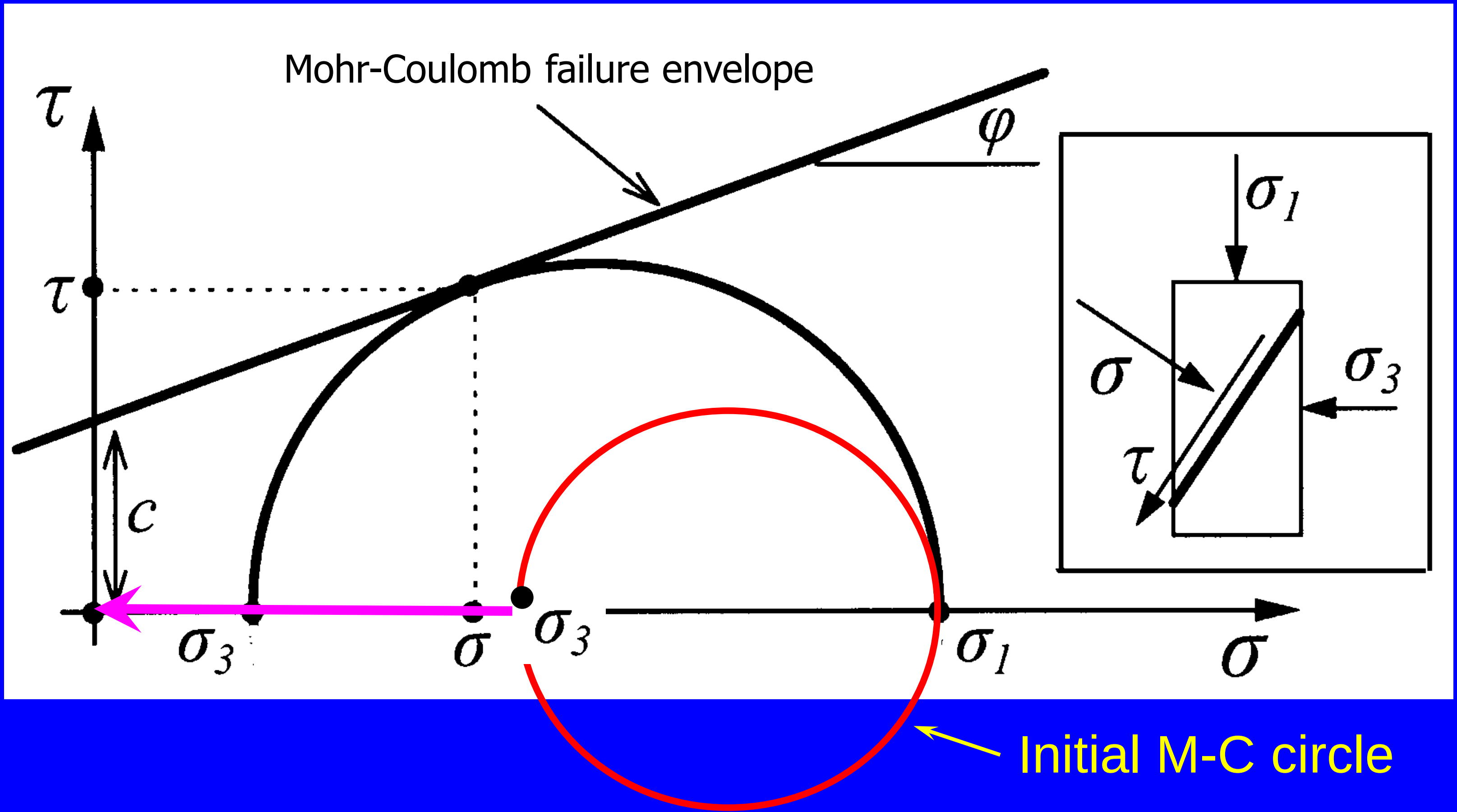
$$x = \frac{1}{2} H \tan\left(45 - \frac{\phi}{2}\right)$$

2 x = width of the top of the wedge
H = tunnel height
φ = ground friction angle

Analysis of Tunnel Face Stability

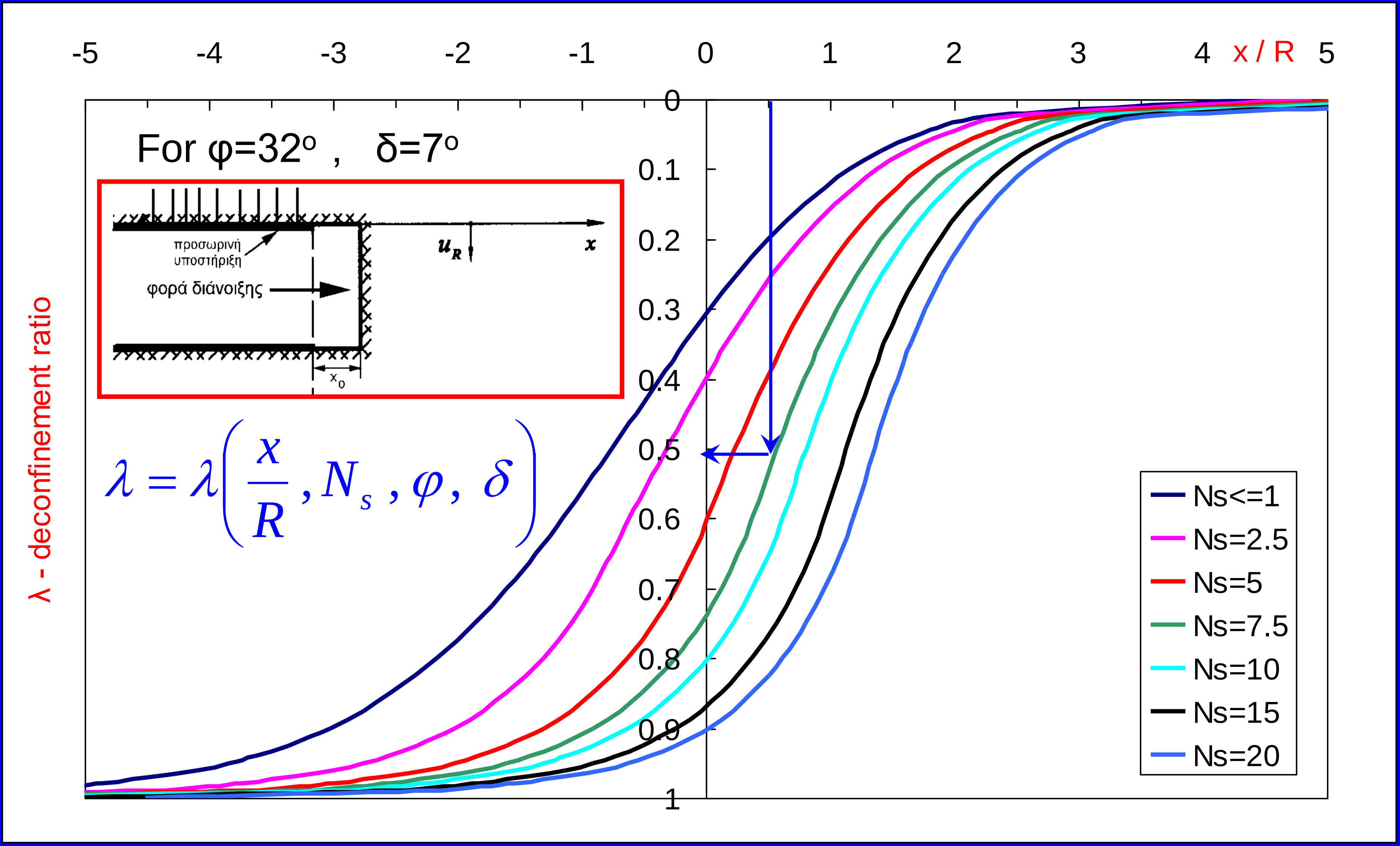
Risk of tunnel face failure (instability) increases with:

- Reduction of ground strength (σ_{cm})
- Increase of tunnel depth (i.e., increase of σ_1)
- Size of the tunnel face (reduction of 3D effects, favourable in stability)
- Hydraulic flow gradient towards tunnel face



Analysis of Tunnel Face Stability

Value of the deconfinement factor (λ) at distance (x) ahead of the tunnel face:



For φ=32°, δ=7°

$$\lambda = \lambda\left(\frac{x}{R}, N_s, \varphi, \delta\right)$$

Analysis of Tunnel Face Stability – Unsupported tunnel face

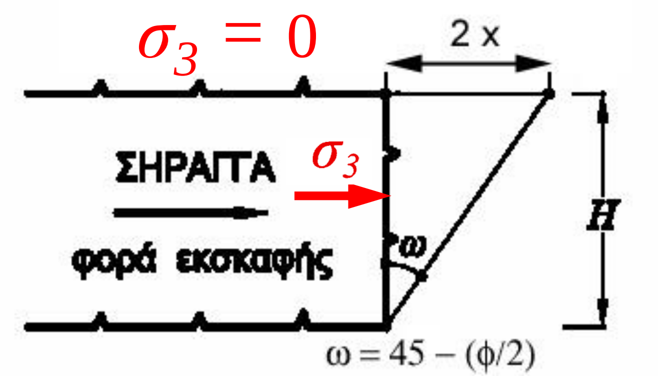
At the tunnel face: $\sigma_3 = 0$ (i.e., under uniaxial stress σ_1)

Factor of safety against tunnel face instability:

$$FS_o = \frac{strength}{stress} = \frac{\sigma_{cm}}{\sigma_1}$$

where: $$\sigma_{cm} = \frac{\sigma_{ci}}{50} \exp\left(\frac{GSI}{25.5}\right)$$

$\sigma_1 = (1-\lambda)\, p_o$ = vertical stress at distance (x) ahead of tunnel face, using the deconfinement factor (λ) instead of the "Terzaghi silo theory", where:



$$x = \frac{1}{2} H \tan\left(45 - \frac{\phi}{2}\right)$$

2 x = width of the top of the wedge

H = tunnel height

φ = ground friction angle

Above formula gives: $$\boxed{FS_o = \frac{2}{(1-\lambda)N_s}}$$

where: $$N_s = \frac{2\,p_o}{\sigma_{cm}}$$

Analysis of Tunnel Face Stability – Unsupported tunnel face

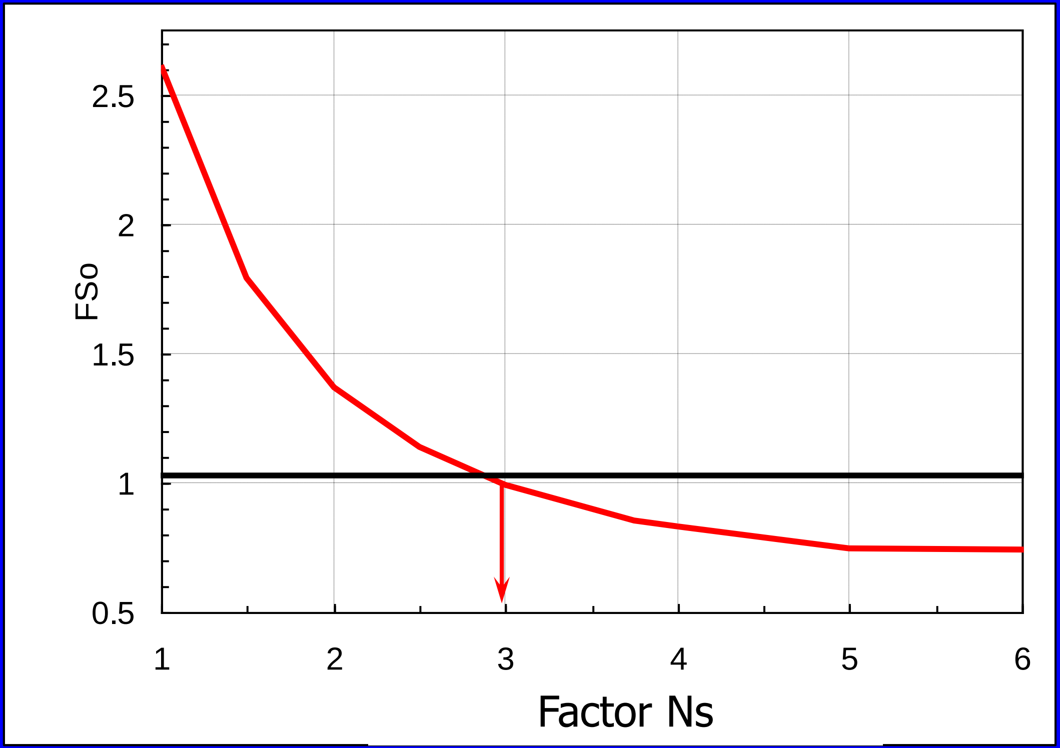
Factor of safety against tunnel face instability:

$$\boxed{FS_o = \frac{2}{(1-\lambda)N_s}} \quad \text{where:} \quad N_s = \frac{2\,p_o}{\sigma_{cm}}$$

Values of FS_o for $x / R = 1 / 3$, tunnel radius R=5.5m and 2x = 3.5m) :

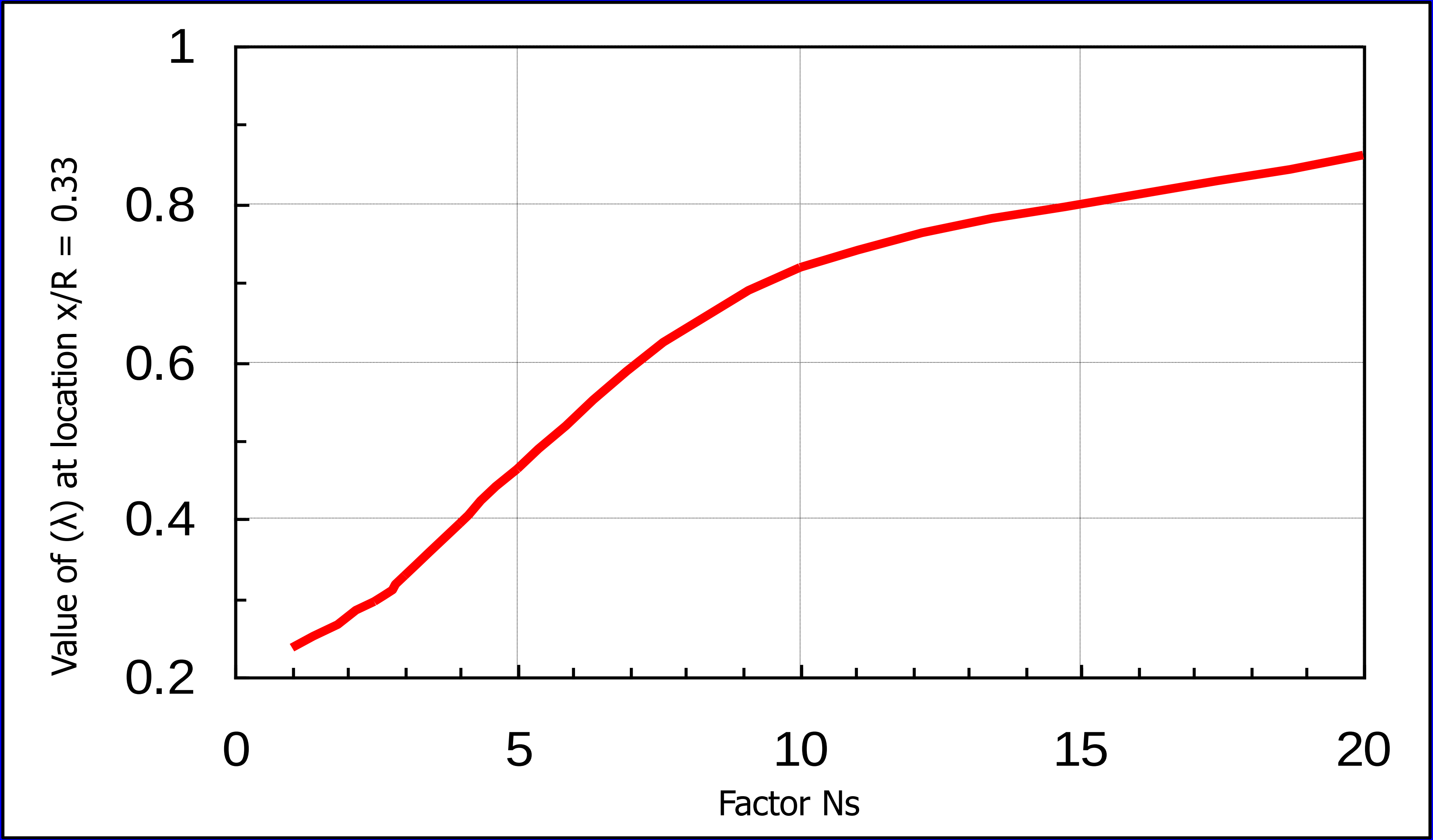
Face is stable for $N_s \leq 3$

N_s	**λ**	**FS_o**
< 1	0.235	2.61
2.5	0.295	1.13
3	**0.327**	**1.00**
4	0.395	0.83
5	0.462	0.74
10	0.720	0.71
20	0.860	0.71



Analysis of Tunnel Face Stability – Unsupported tunnel face

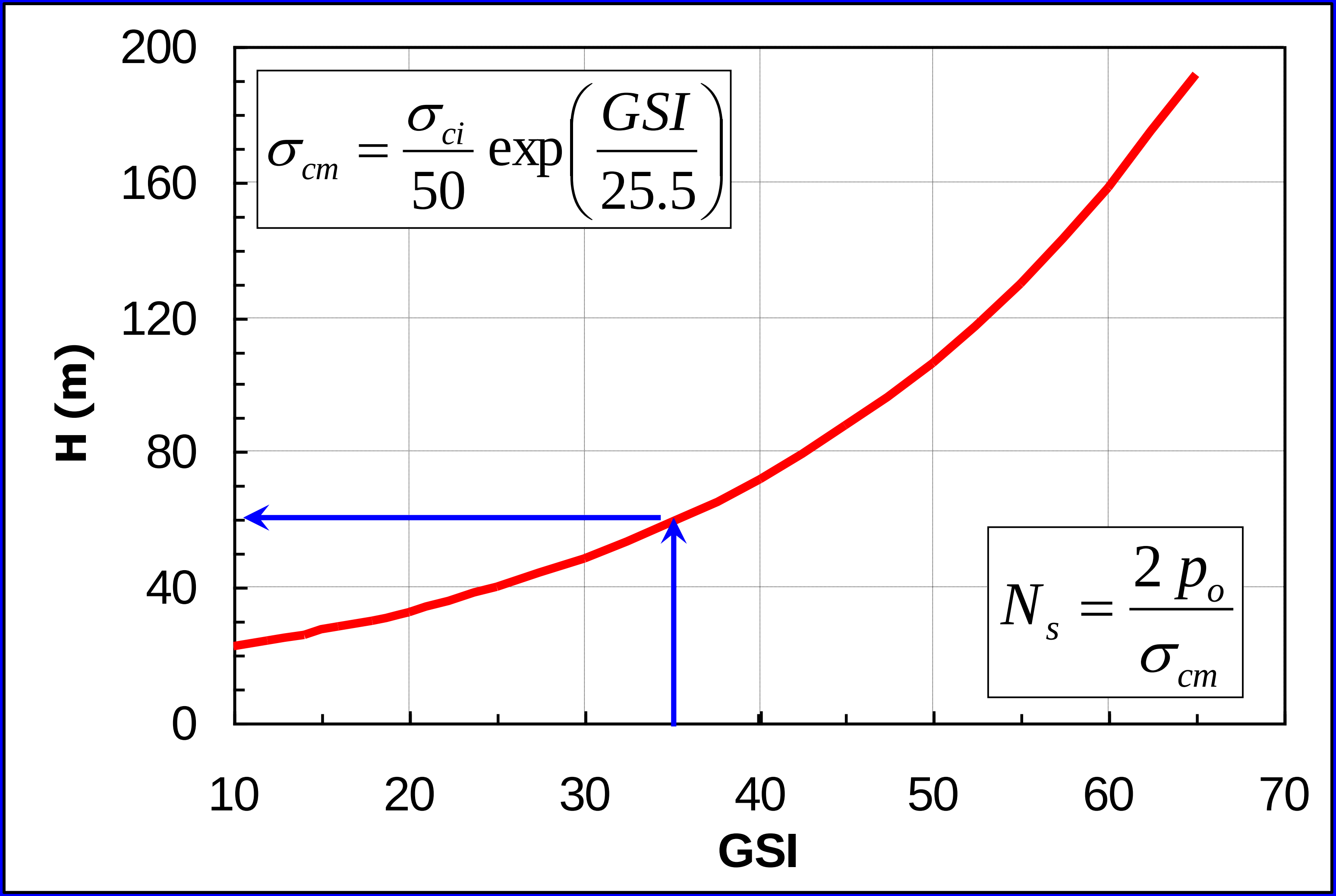
Approximate values of (λ) at x/R=0.33 (common location of support installation) in terms of N_s :



Analysis of Tunnel Face Stability – Unsupported tunnel face

Maximum tunnel depth (H) where tunnel face remains stable (with FS=1, i.e., where $N_s \leq 3$), in terms of rockmass index GSI:

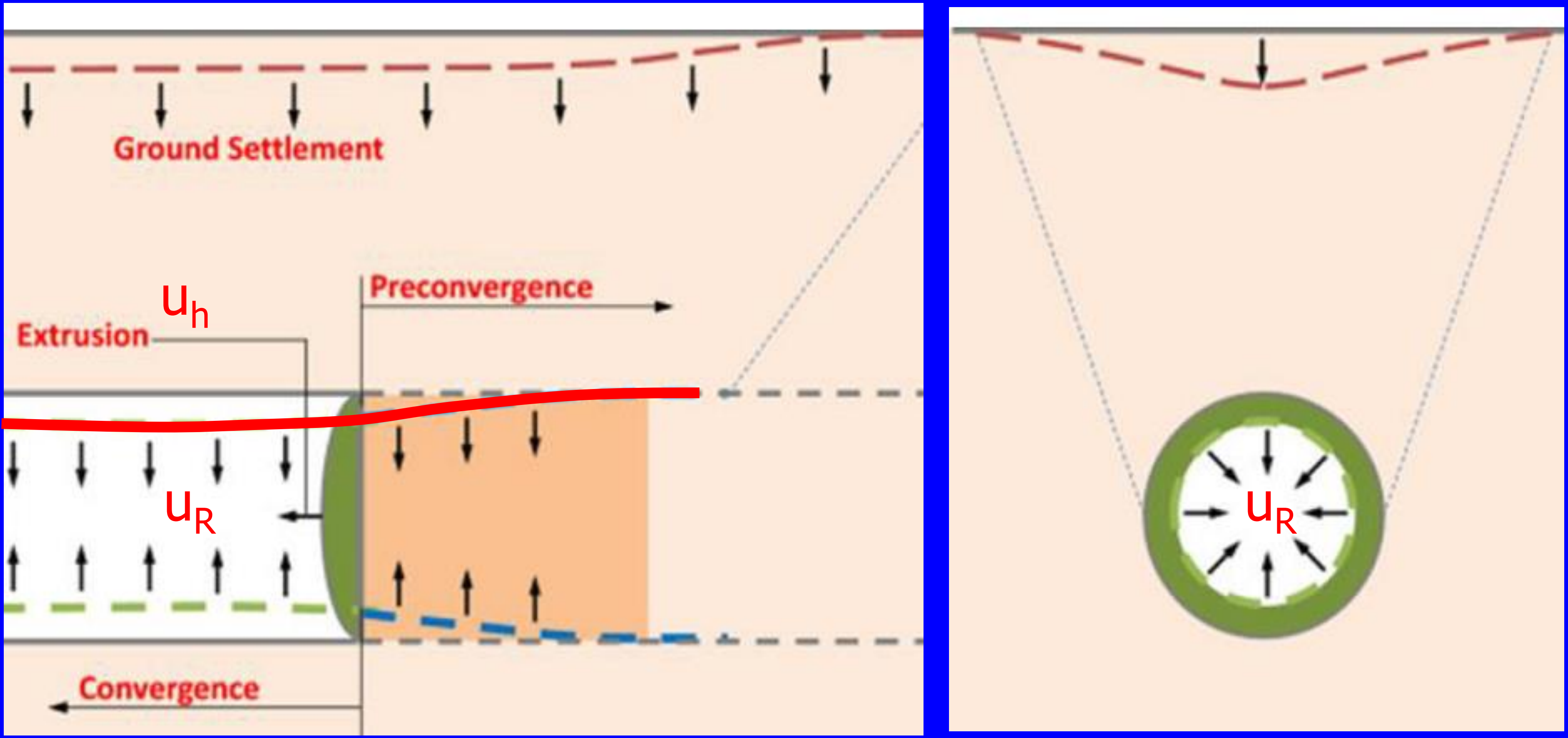
(for $\sigma_{ci} = 12$ MPa , $\gamma = 24$ kN/m^3)



$$\sigma_{cm} = \frac{\sigma_{ci}}{50} \exp\left(\frac{GSI}{25.5}\right)$$

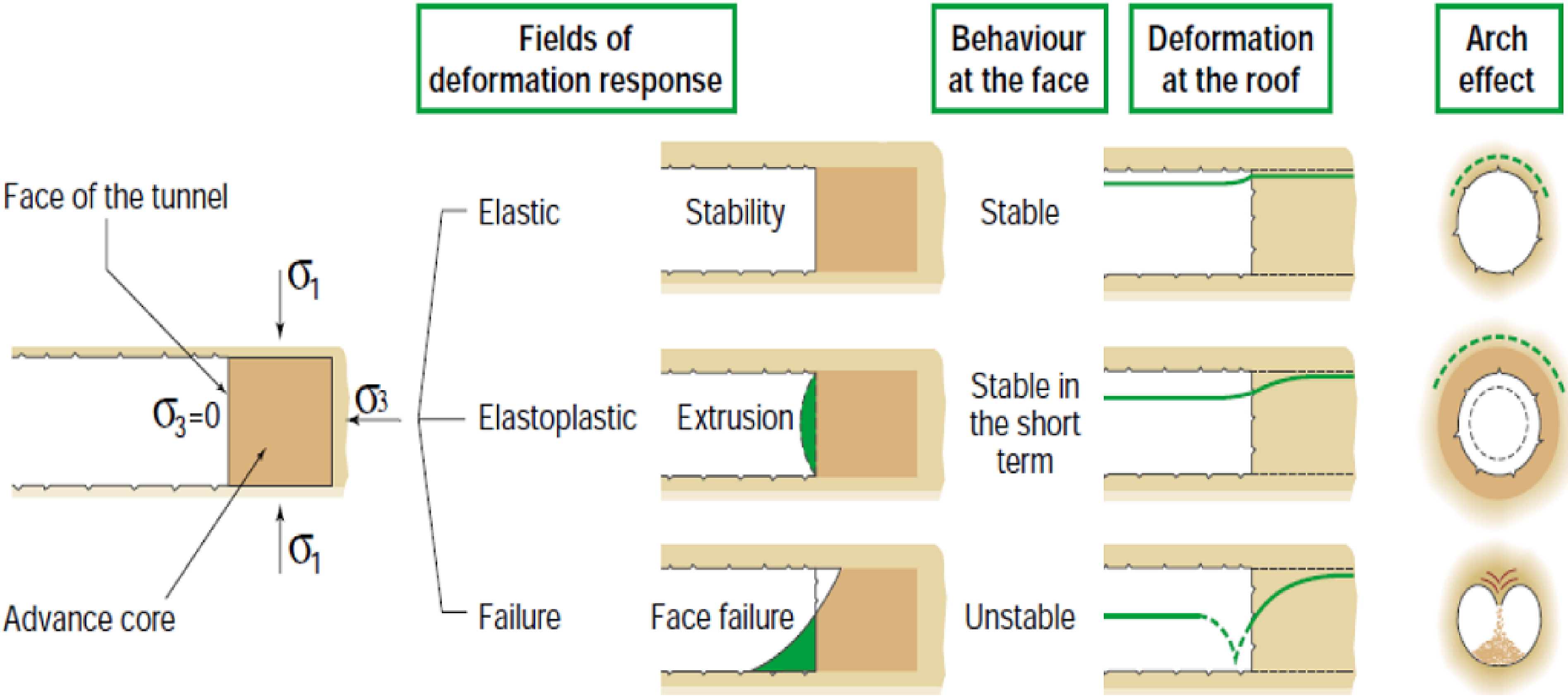
$$N_s = \frac{2\,p_o}{\sigma_{cm}}$$

Analysis of Tunnel Face Stability – Unsupported tunnel face



U_h = average inward displacement of tunnel face (extrusion)

Analysis of Tunnel Face Stability – Unsupported tunnel face



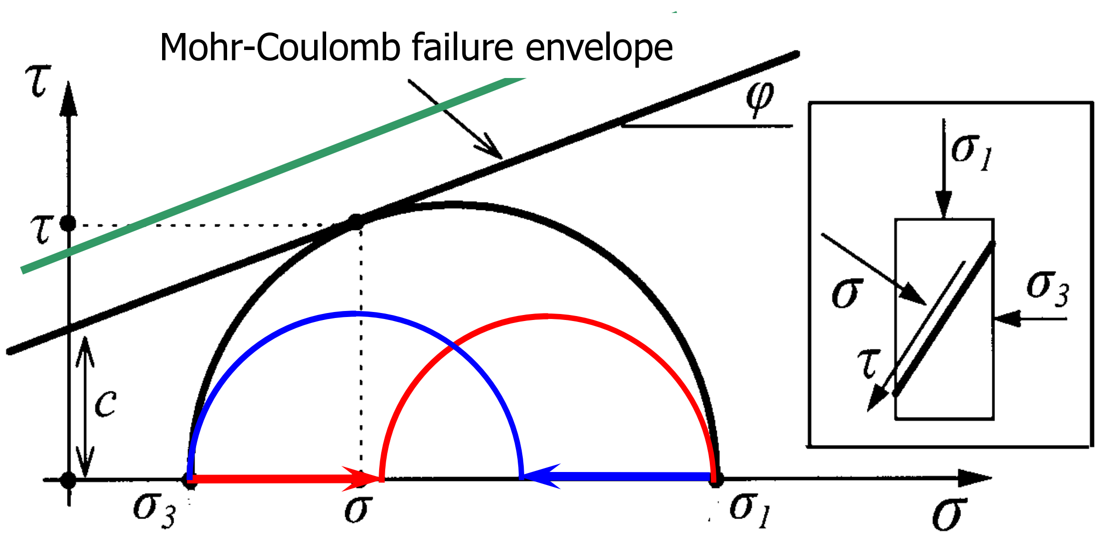
Types of tunnel face behaviour (Lunardi, 2000):

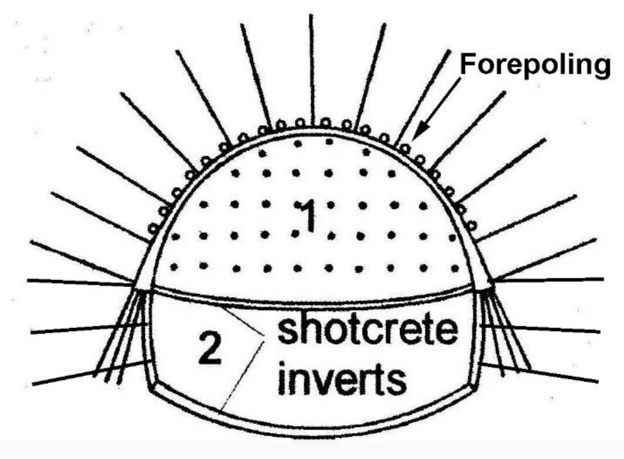
1. Elastic - Stable (Ns < 1)
2. Elasto-plastic - Stable (Ns ≈ 1 ÷ 3)
3. Unstable (Ns > 3 about)

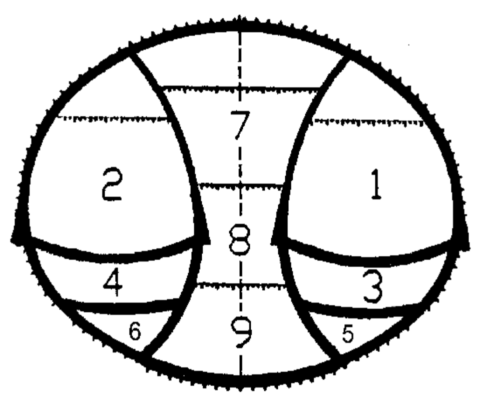
Analysis of Tunnel Face Stability – Supported tunnel face

When the unsupported tunnel face is unstable, face stability can be improved by the following methods:

1. Increase of σ_3 (red)
2. Reduction of σ_1 (blue)
3. Increase of ground strength (c, φ) (green)
4. Reduction of tunnel face area
5. Reduction of groundwater pressure



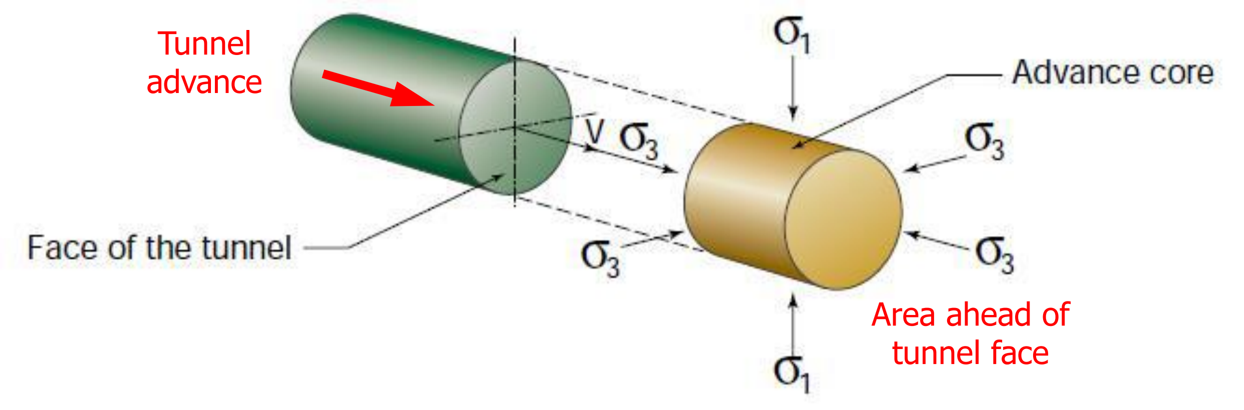


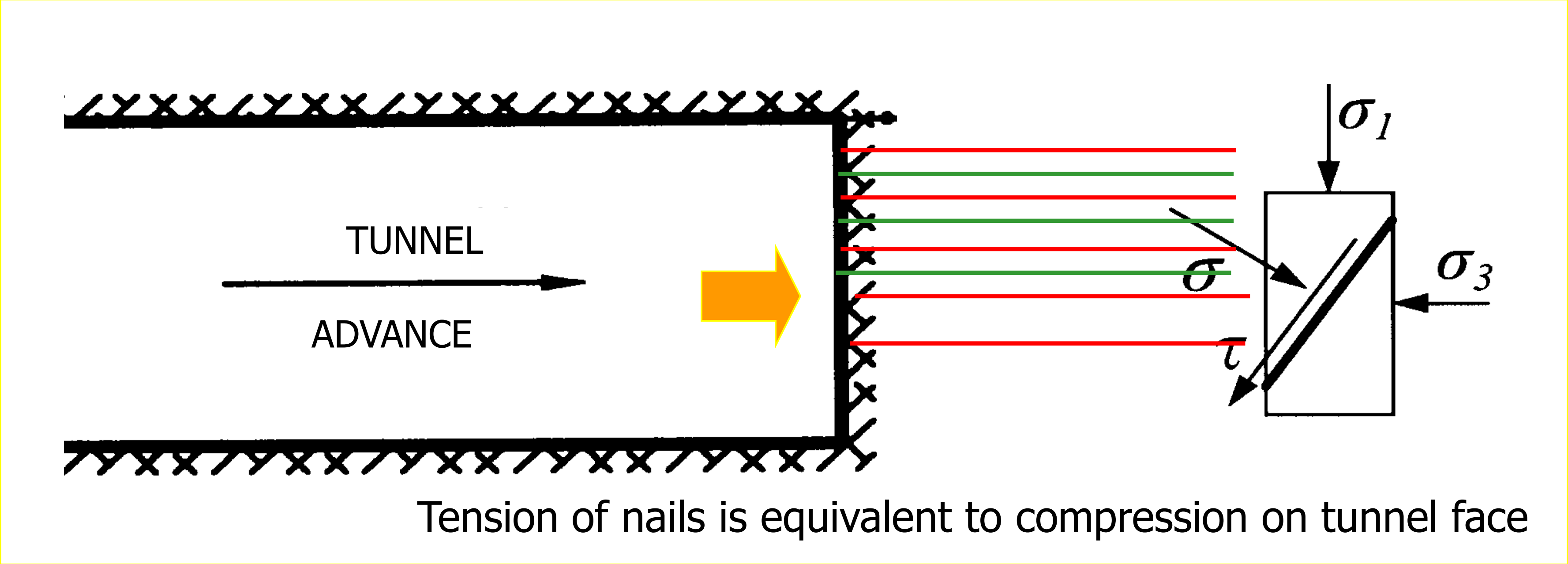


Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

1. Increase of σ_3 :
   - Face reinforcement with Fiberglass nails





Tension of nails is equivalent to compression on tunnel face

Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

1. Increase of σ_3 : Face reinforcement with Fiberglass nails

Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

1. Increase of σ_3 : Face reinforcement with Fiberglass nails

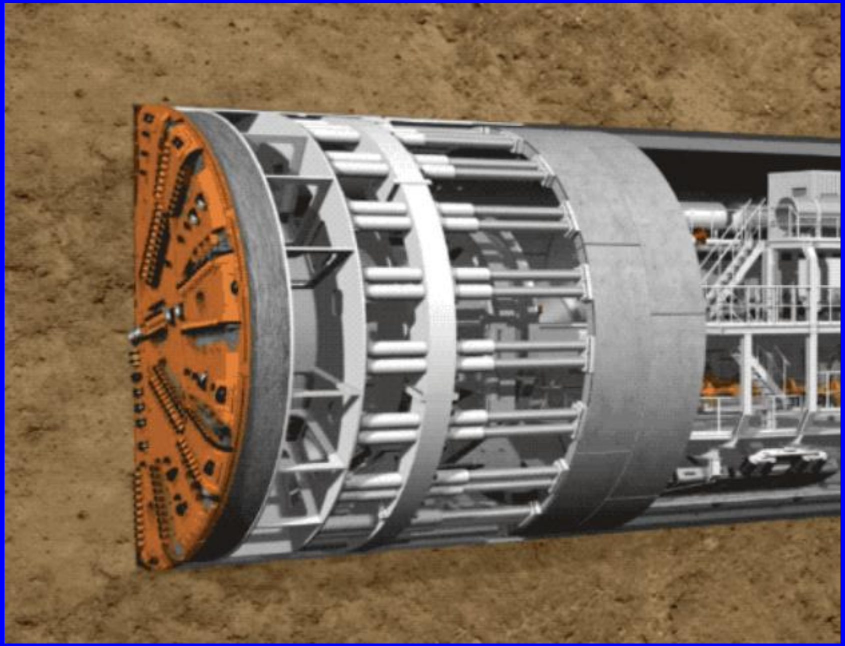
Installation of Fiber-Glass nails on tunnel face

Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

1. Increase of σ_3 :

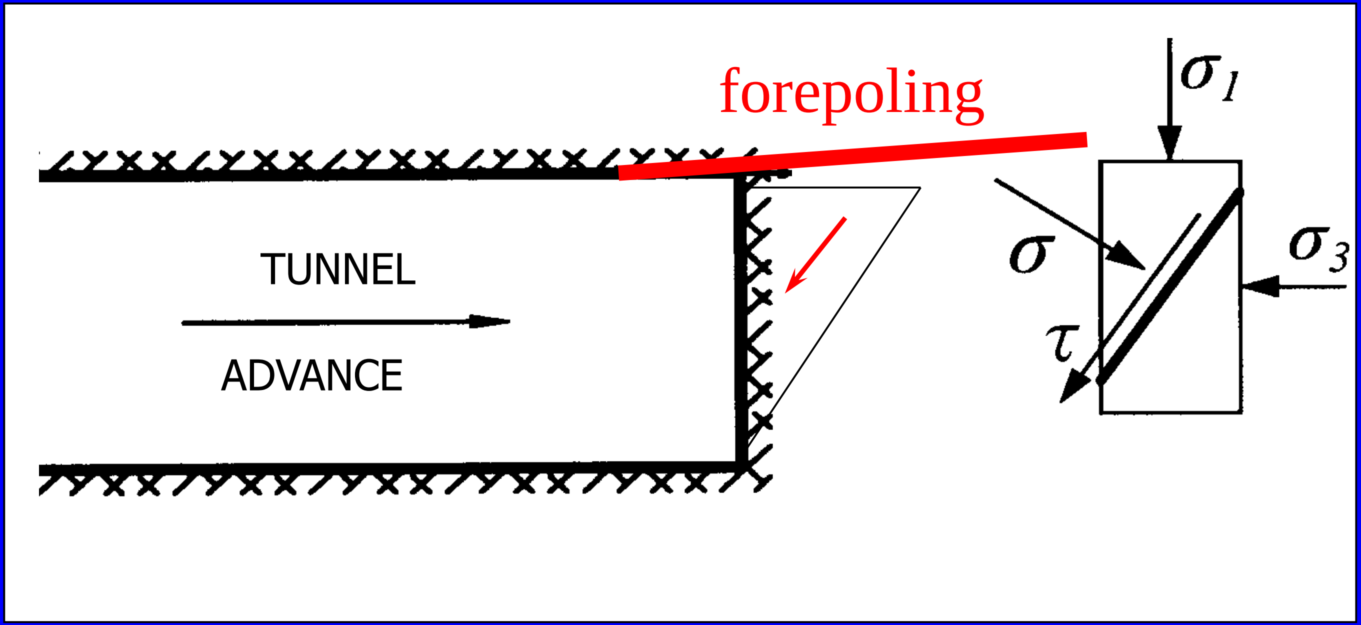
   The TBM maintains increased pressure (σ_3) on tunnel face

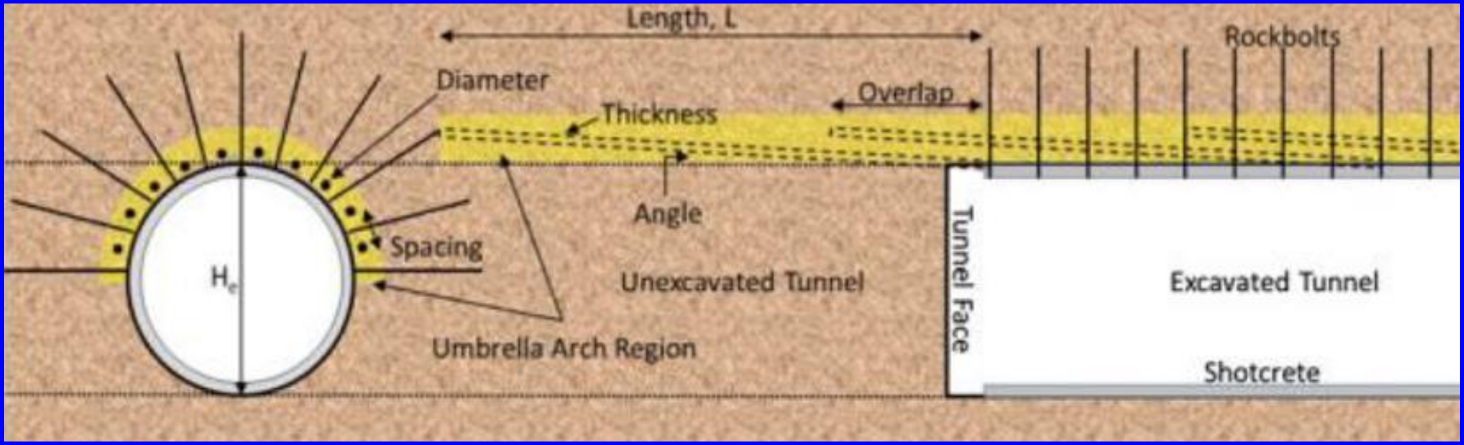


Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

2. Reduction of σ_1 : Use of forepoling

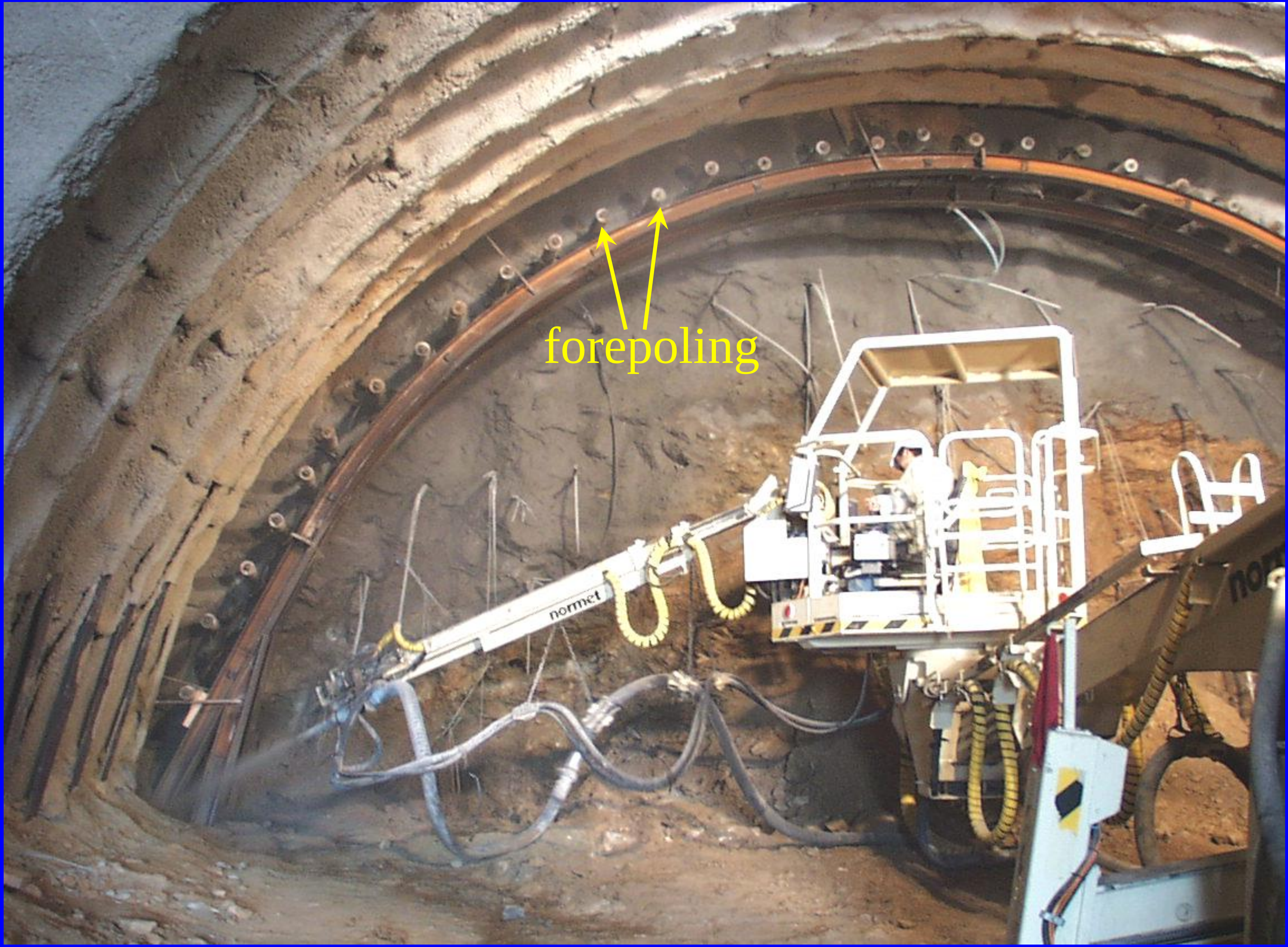




Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

2. Reduction of σ_1 : Use of forepoling



Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

2. Reduction of σ_1 : Use of forepoling

Analysis of Tunnel Face Stability – Supported tunnel face

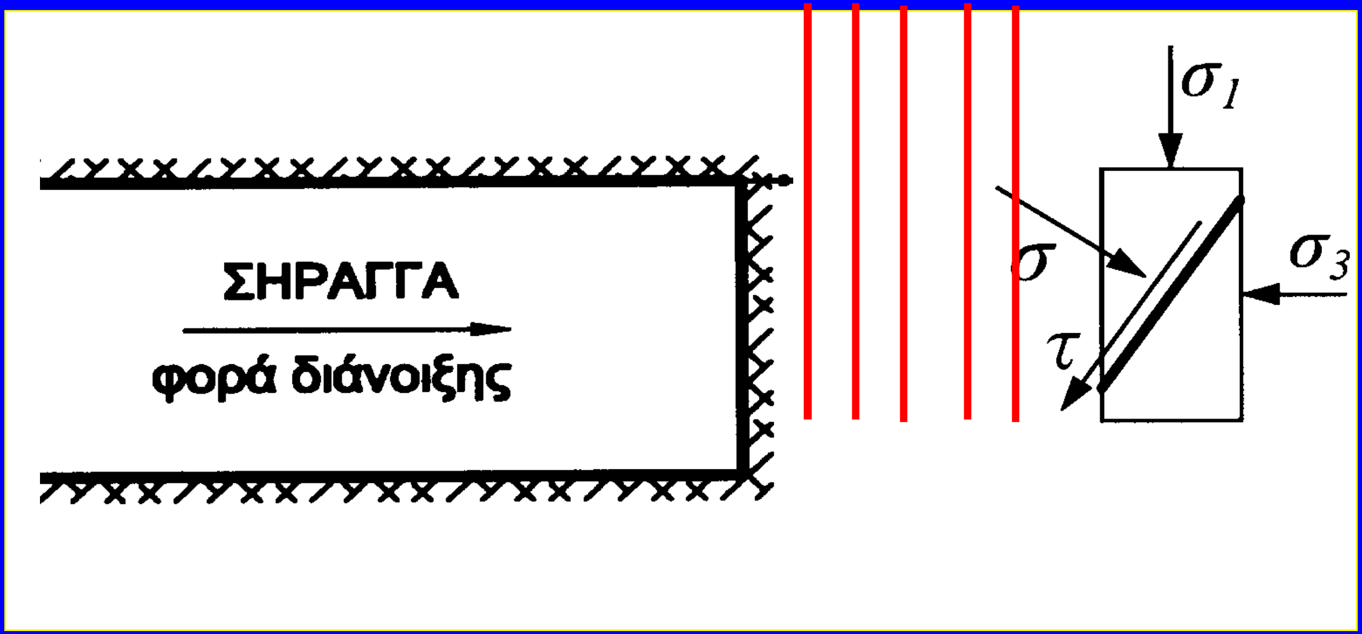
Methods to improve tunnel face stability:

2. Reduction of $\sigma_{\mathbf{1}}$: Use of forepoling

Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

2. Reduction of σ_1 : Use of vertical Fiber-Glass nails, installed from ground surface (in shallow tunnels only)

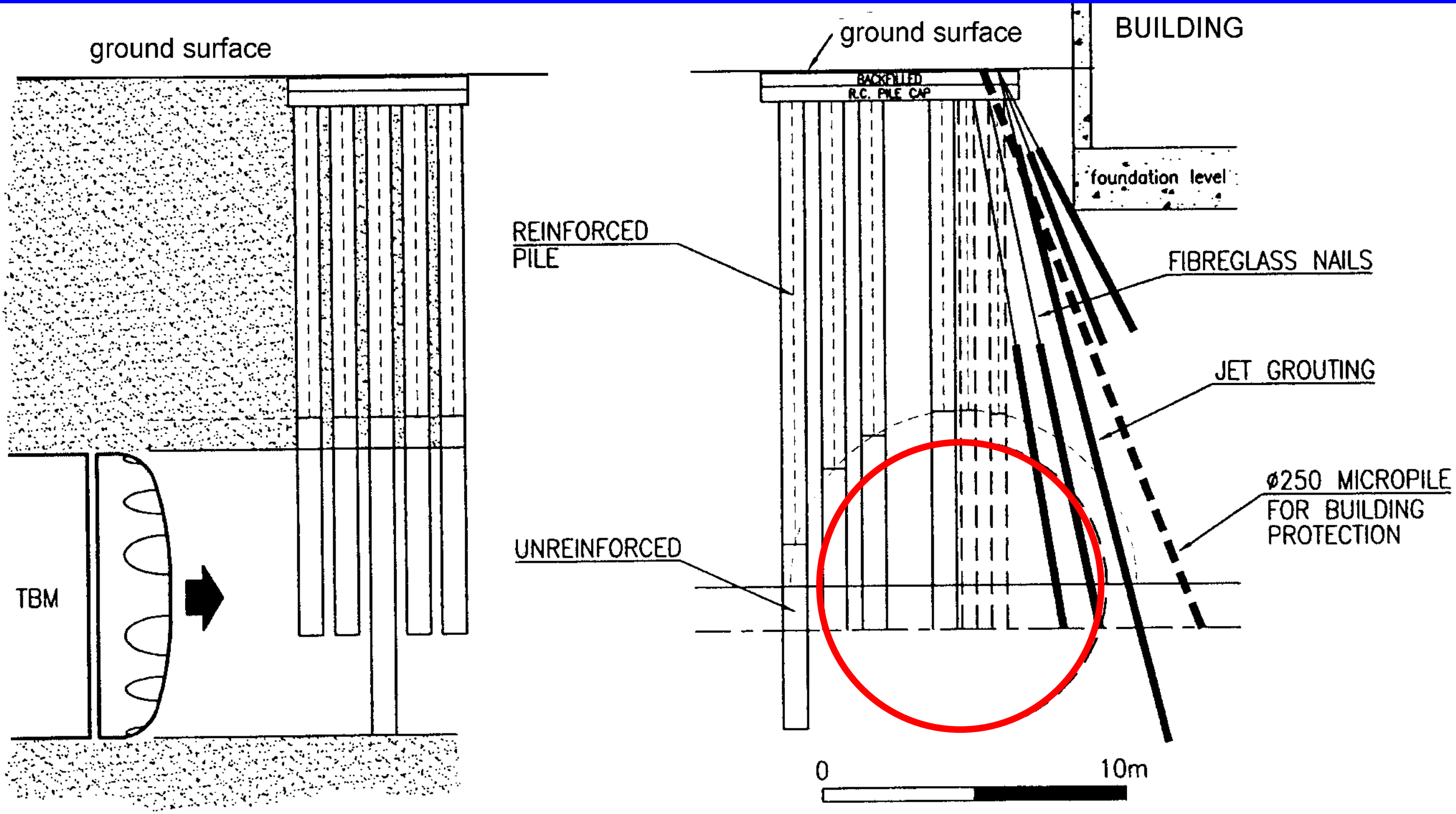


Tension in the FG nails supports the ground and reduces σ_1

Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

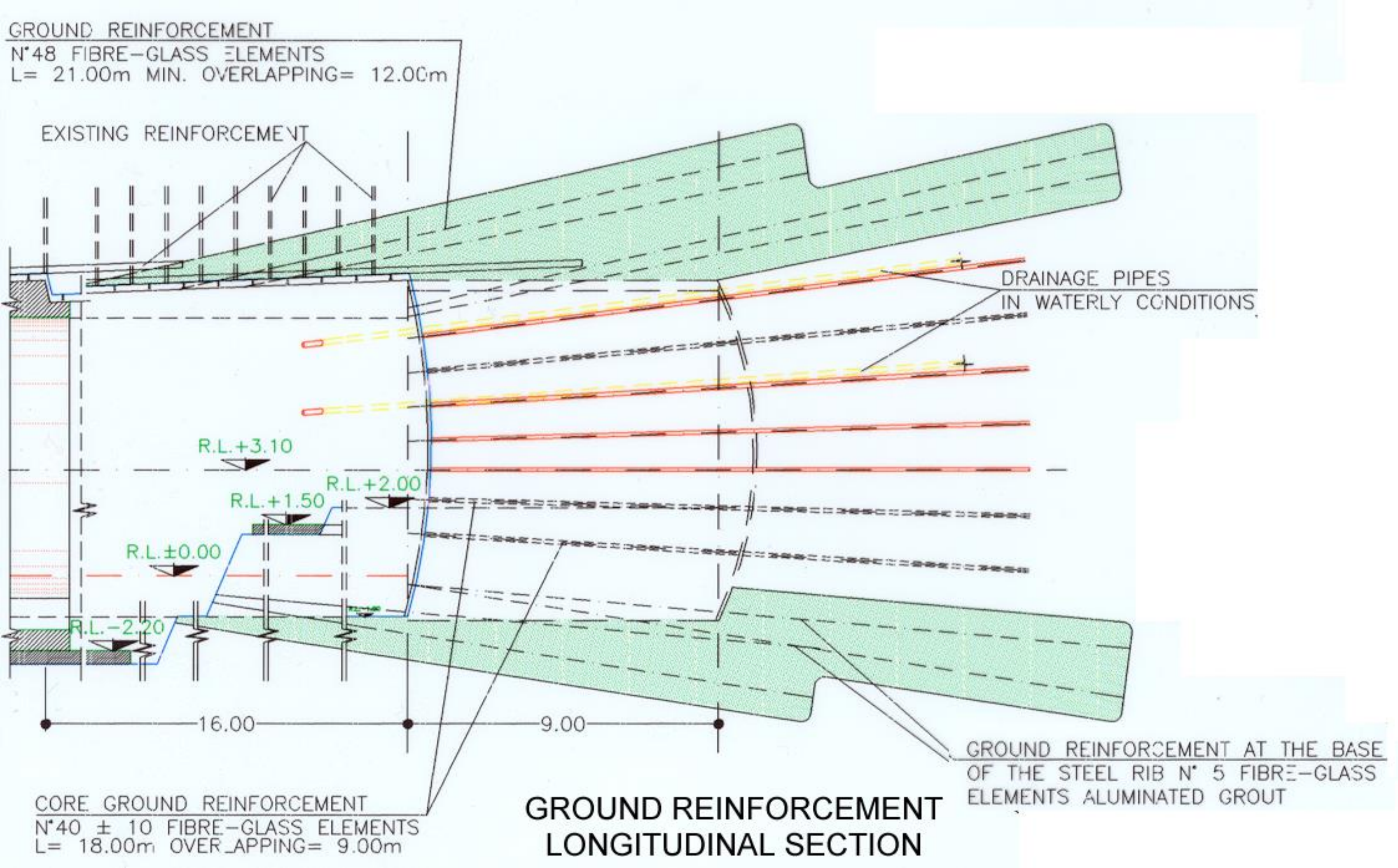
2. Reduction of σ_1 : Use of vertical Fiber-Glass nails, installed from ground surface (in shallow tunnels only)



Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

2. Reduction of σ_1 : Use of an umbrella of contiguous jet-grouted columns – they cause arching across the tunnel section, reducing σ_1

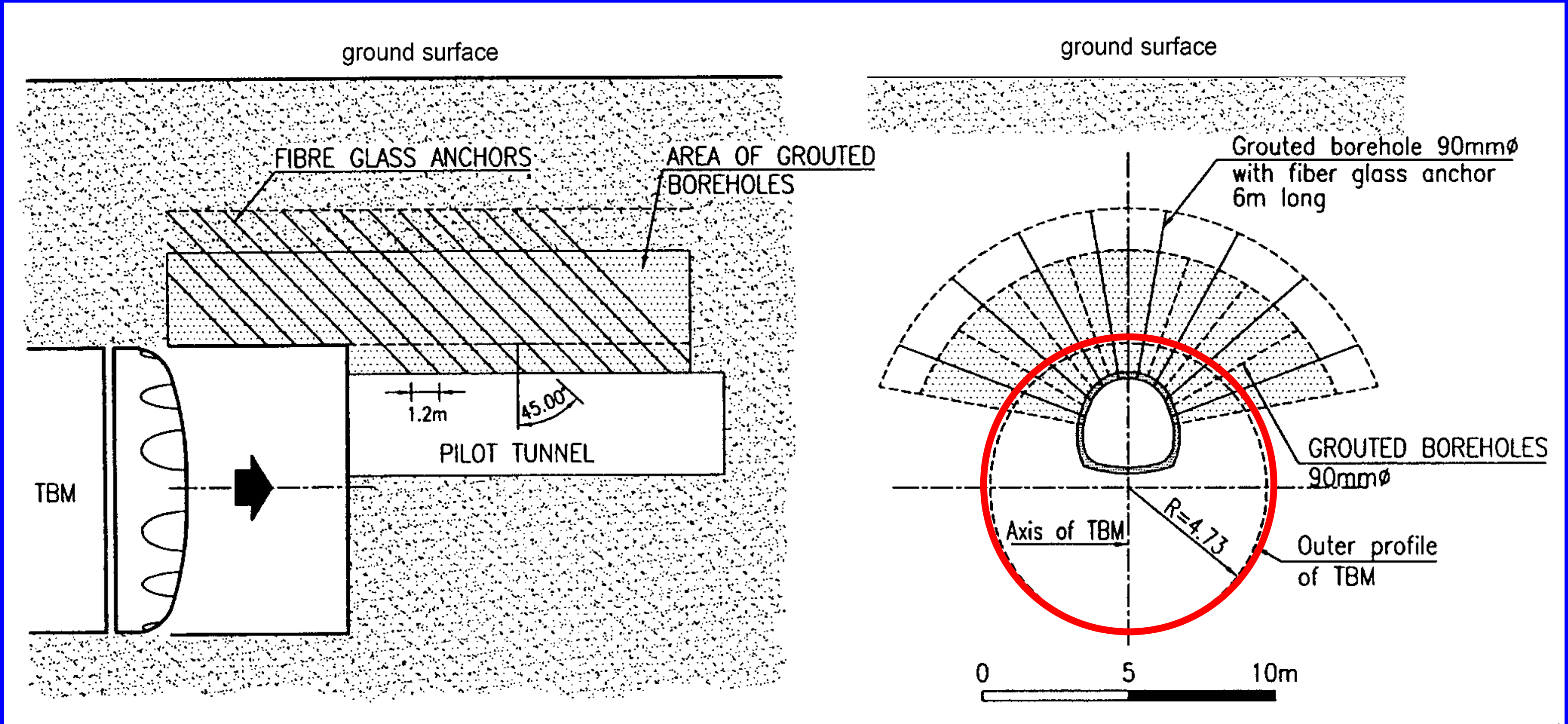


GROUND REINFORCEMENT
LONGITUDINAL SECTION

Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability:

3. Increase ground strength ahead of tunnel face:
   - Grouting
   - Ground freezing
   - Dewatering (to reduce pore water pressures)

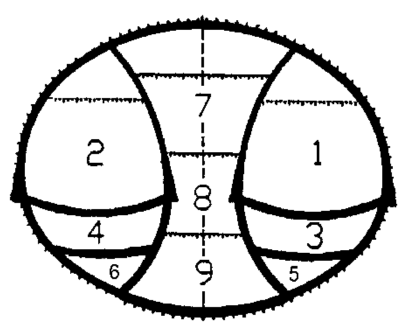


Analysis of Tunnel Face Stability – Supported tunnel face

Methods to improve tunnel face stability :

4. Reduction of crest raveling using spilling

5. Reduce the size of the tunnel face with multi-stage excavation



Analysis of Tunnel Face Stability – Support with forepoling

(a) Safety factor of the unsupported tunnel face:

$$\boxed{FS_o = \frac{2}{(1-\lambda)N_s}}$$

where :

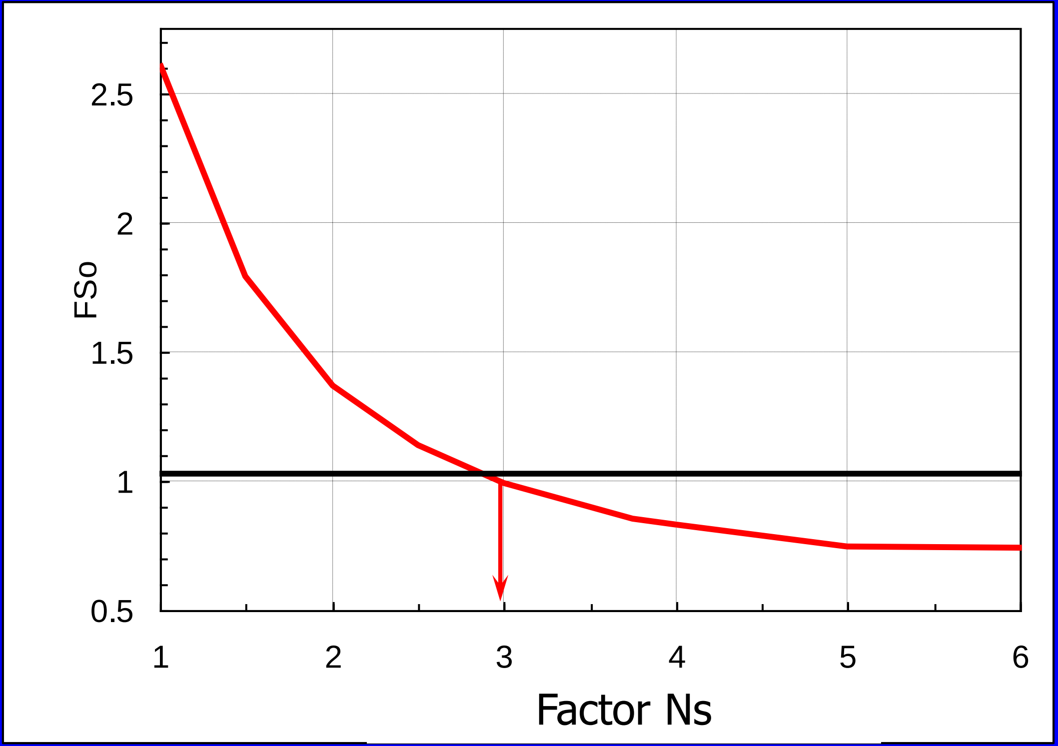
$\sigma_1 = (1-\lambda)\, p_o$ (λ at location x)

$$N_s = \frac{2\,p_o}{\sigma_{cm}}$$

Values of the safety factor FS_o for x / R = 1 / 3 and R=5.5m, 2x = 3.5m) :

Face is stable for $N_s \leq 3$

N_s	**λ**	**FS_o**
< 1	0.235	2.61
2.5	0.295	1.13
3	**0.327**	**1.00**
4	0.395	0.83
5	0.462	0.74
10	0.720	0.71
20	0.860	0.71



Analysis of Tunnel Face Stability – Support with forepoling

(b) Safety factor of a tunnel face supported with forepoling:

If the vertical pressure undertaken by forepoling is p_f then:

$$FS = \frac{strength}{pressure} = \frac{\sigma_{cm}}{\sigma_1 - p_f} = \frac{2}{N_s\left[(1-\lambda) - \dfrac{p_f}{p_o}\right]} = \frac{FS_o}{1 - \left(\dfrac{p_f}{p_o}\right)\dfrac{1}{(1-\lambda)}}$$

where : $N_s = \dfrac{2\,p_o}{\sigma_{cm}}$ $\sigma_1 = (1\text{-}\lambda)\,p_o$

N_s	λ	p_f / p_o
≤ 3	< 0.328	0
3.75	0.375	0.092
4	0.395	0.105
5	0.462	0.138
7.5	0.63	0.103
10	0.723	0.077
15	0.814	0.053
20	0.86	0.040

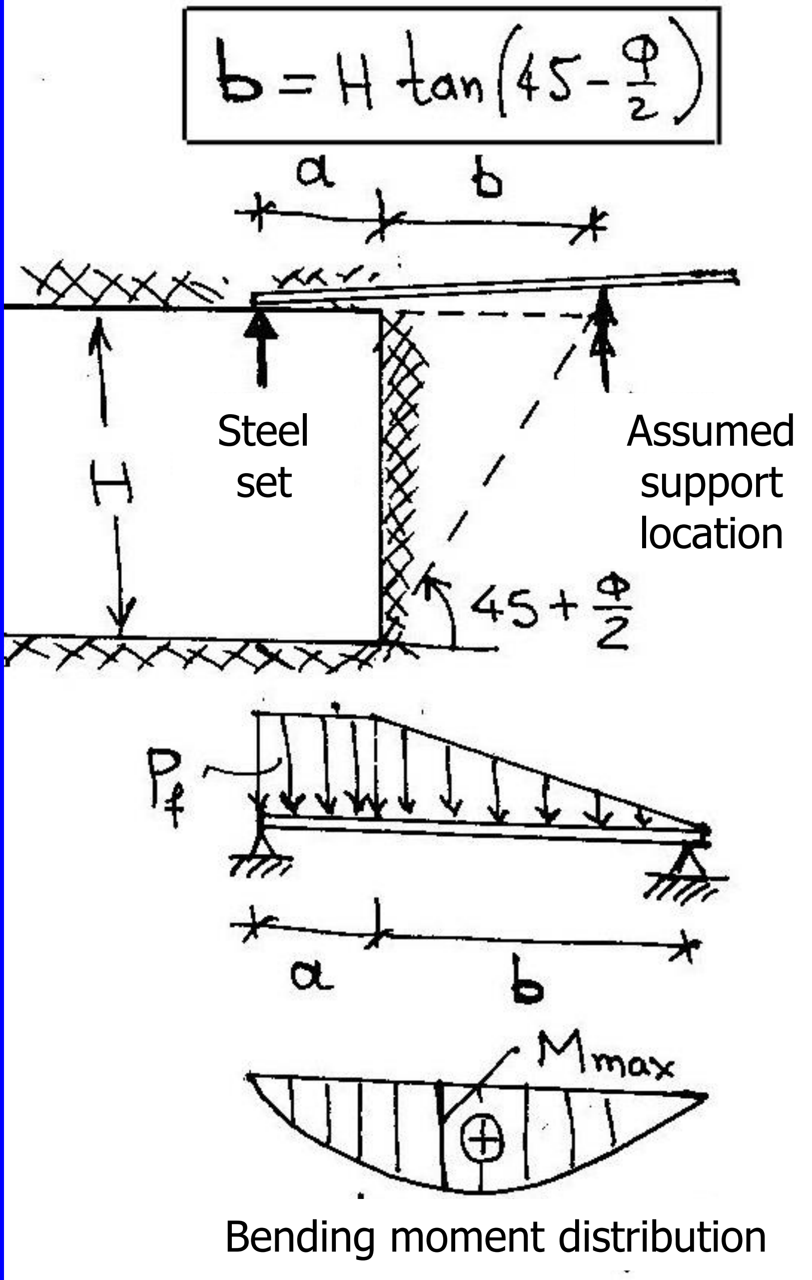
p_f values to achieve safety factor FS=1 (for x / R = 1/3) :

p_f values to achieve safety factor FS :

$$\boxed{\frac{p_f}{p_o} = (1-\lambda)\left(1 - \frac{FS_o}{FS}\right)}$$

$$FS_o = \frac{2}{(1-\lambda)N_s}$$

Analysis of Tunnel Face Stability – Support with forepoling



Bending moment distribution

(c) Calculation of the maximum pressure (p_f) that forepoling can undertake

The maximum pressure (p_f) corresponds to the maximum bending moment M_{max} of the forepoles (usually allowed to reach the yield value M_y), using the shown assumption about the ground pressure distribution along the forepoles, their section modulus and spacing.

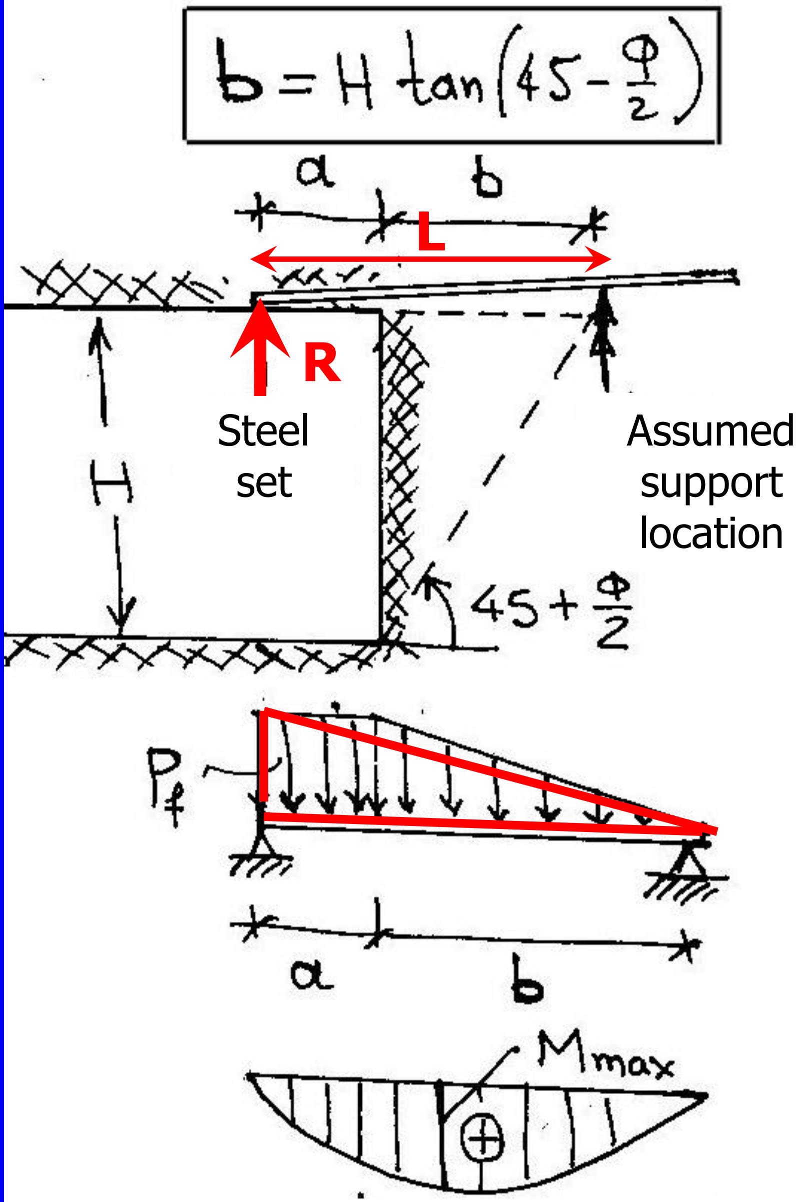
NOTE: Steel sets (with good base support – elephant foot) are absolutely necessary with forepoling.

p_f = ground pressure distribution along the forepole

a = distance of last support steel set from the tunnel face (about 1m)

b = width of failing ground wedge
$b = H \tan(45 - \varphi/2)$

Analysis of Tunnel Face Stability – Support with forepoling



Bending moment distribution

(c) Calculation of the maximum pressure (p_f) that forepoling can undertake

Approximate triangular pressure distribution on forepole:

p_f = maximum ground pressure
0 = minimum pressure at end of wedge
Loaded forepole length: L = a + b
B = spacing of forepoles

Maximum bending moment on forepole:

$$M_{\max} \approx \frac{\sqrt{3}}{27} p_f \, L^2 \, B$$

The forepoling tubes transfer significant loads at their rear end, on the last steel set. The reaction R is:

$$R = \frac{1}{3} p_f \, L \, B$$

The steel sets and, especially, their foundation should be designed to undertake this force.

Analysis of Tunnel Face Stability – Support with forepoling

(d) Design of forepoling:

1. Calculate the factor of safety of the unsupported tunnel face (FS_o). Usually, the minimum acceptable value is: $FS_{all} = 1.0 – 1.1$. If $FS_o \geq FS_{all}$, the face is stable (no support required).
2. If $FS_o < FS_{all}$ and it is decided to support the tunnel face with forepoles, calculate the required pressure (p_f) to achieve the required factor of safety (with support). Usually $FS = FS_{all}$
3. Calculate the forepole bending moment M_{max} corresponding to (p_f)
4. Select forepoles (and spacing) to undertake M_{max} . Usually, $M_{max} = M_y$ (yield moment of the forepoles). Steel tubes are used as forepoles.
5. Calculate the reaction force (R) of each forepoling tube, and design the steel sets (and their foundation) to undertake these forces

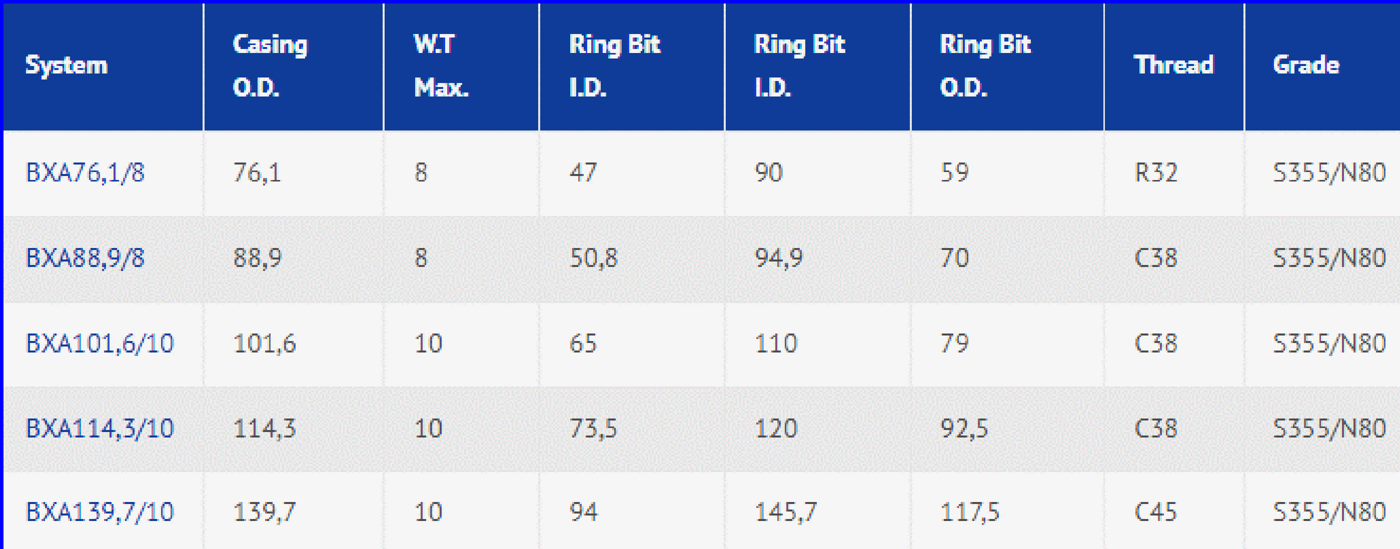
$$\boxed{FS_o = \frac{2}{(1-\lambda)N_s}}$$

$$\boxed{\frac{p_f}{p_o} = (1-\lambda)\left(1-\frac{FS_o}{FS}\right)}$$

$$\boxed{M_{\max} \approx \frac{\sqrt{3}}{27} p_f L^2 B}$$

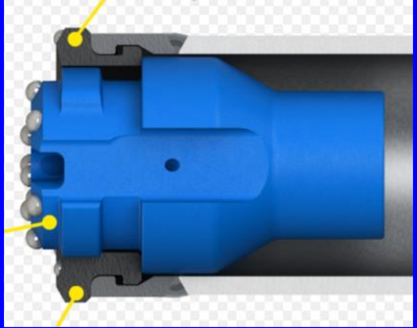
Analysis of Tunnel Face Stability – Support with forepoling

(d) Design of forepoling tubes:



System	Casing O.D.	W.T Max.	Ring Bit I.D.	Ring Bit I.D.	Ring Bit O.D.	Thread	Grade
BXA76,1/8	76,1	8	47	90	59	R32	S355/N80
BXA88,9/8	88,9	8	50,8	94,9	70	C38	S355/N80
BXA101,6/10	101,6	10	65	110	79	C38	S355/N80
BXA114,3/10	114,3	10	73,5	120	92,5	C38	S355/N80
BXA139,7/10	139,7	10	94	145,7	117,5	C45	S355/N80

Yield Strength	Outer Diameter	Wall Thickness	Weight	Section Modulus	Second Moment of Area
[N/mm^2]	[mm]	[mm]	[kg/m]	[cm^3]	[cm^4]
355	76.1	6.3	10.8	22	85
	88.9	5.0	10.4	26	116
	88.9	**6.3**	**12.8**	**32**	**140**
	88.9	8.0	16.0	38	168
	114.3	5.0	13.5	45	257
	114.3	**6.3**	**16.8**	**55**	**313**
	114.3	8.0	21.0	66	379
	139.7	5.0	16.6	69	481
	139.7	**6.3**	**20.7**	**84**	**589**
	139.7	**8.0**	**26.0**	**103**	**720**
	139.7	10.0	32.0	123	862
	168.3	10.0	39.0	186	1,564
	168.3	12.5	48.0	222	1,868
	168.3	16.0	60.1	267	2,244



Analysis of Tunnel Face Stability – Support with forepoling

Example:

GSI=35, σ_{ci} = 12 MPa, φ = 32°, p_o = 75m x 0.024 = 1.8 MPa

Thus: σ_{cm} = 0.95 MPa , N_S = 3.8 ⇒ λ = 0.38 ⇒ FS_o = 0.85

Face is unstable without support

$$\boxed{FS_o = \frac{2}{(1-\lambda)N_s}}$$

Tunnel height: H = 6m ⇒ b = 3.35m and a = 1m ⟹ L = a + b = 4.35m

Required pressure (p_f) for limiting face stability (FS=1) :

$$\boxed{\frac{p_f}{p_o} = (1-\lambda)\left(1-\frac{FS_o}{FS}\right)}$$

$$p_f / p_o = 0.093 \Rightarrow p_f = 0.093 \times 1800 = 167 \ kPa$$

Bending moment for B=0.45m: M_{max} = 91.3 kNm

$$\boxed{M_{\max} \approx \frac{\sqrt{3}}{27} p_f L^2 B}$$

Required section modulus (W) of forepoling tube (steel S355):

W = M / σ_y = 91.3 / 0.355 = 257.1 cm^3, i.e., the maximum tube Φ168.3 / 16mm @45cm

Reaction force (applied on the steel sets):

R = 0.33 p_f LB = 107.9 kN per forepole = 240 kN / m → p = 240 kPa (for sets @1m)

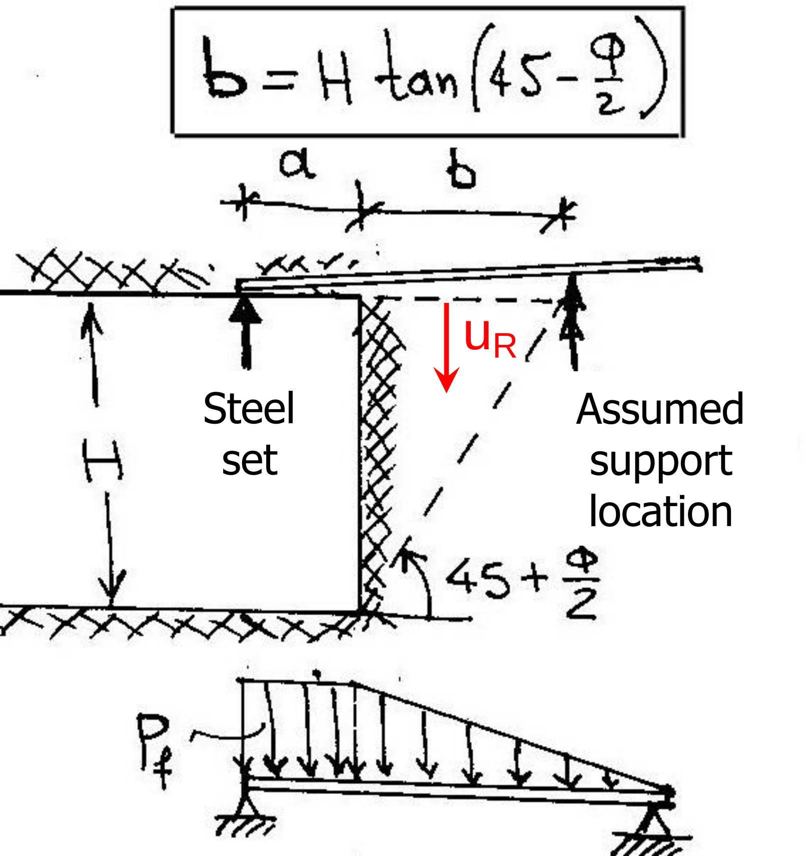
Conclusion: Even at a moderate tunnel depth (75m), face support with forepoling requires very strong forepoling tubes.

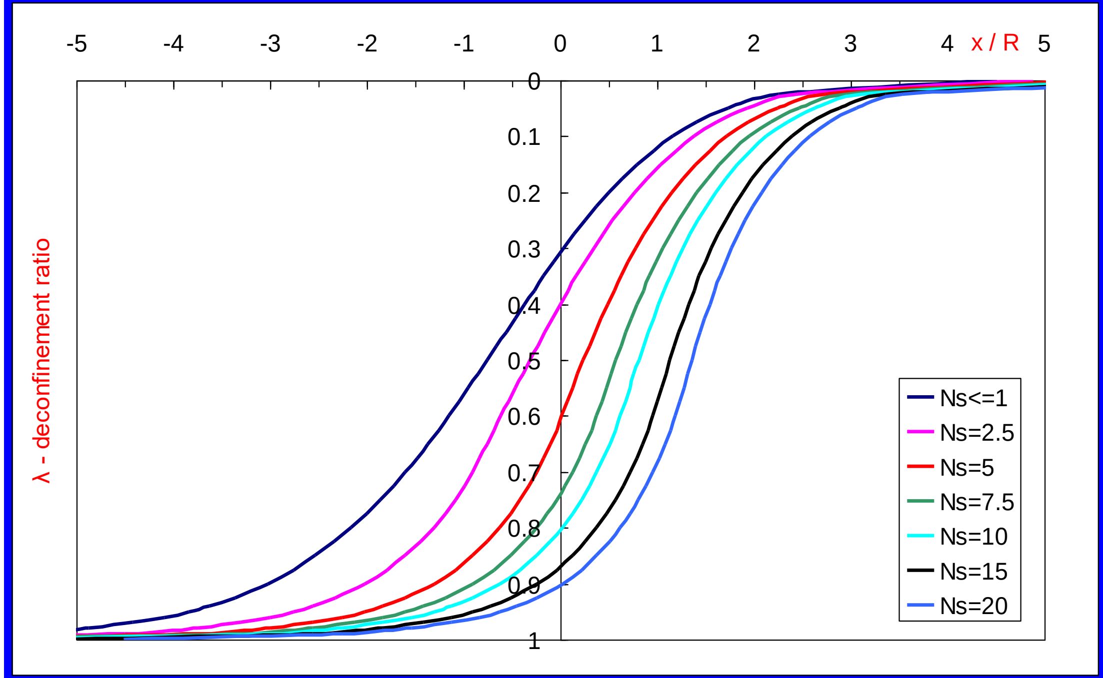
So, forepoling is suitable for small tunnel depths, up to 25m (usually close to the tunnel portals) where p_f is less than 45 kPa, and common forepoles Φ114.3 / 8mm, @45cm spacing can be used. For larger tunnel depths, face support with Fiber-Glass nails is more effective (see following slides).

Analysis of Tunnel Face Stability – Support with forepoling

Forepoling is also used in shallow urban tunnels, to reduce ground surface settlement during NATM tunnelling. In such cases, stiff forepoles reduce u_R ahead of the tunnel face (compared to the u_R without forepoles), thus reducing the deconfinement coefficient (λ) at the tunnel face.

Reduced deconfinement coefficient (λ) means that the steel sets and shotcrete shell will undertake larger support pressure σ_s



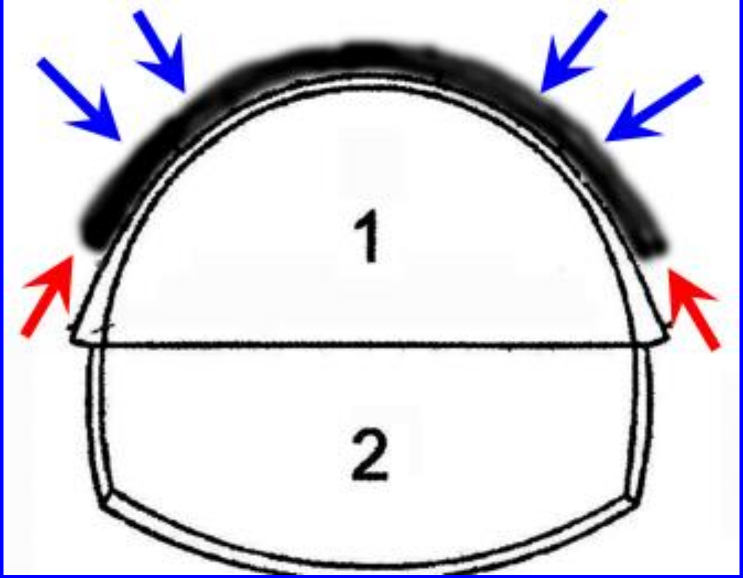


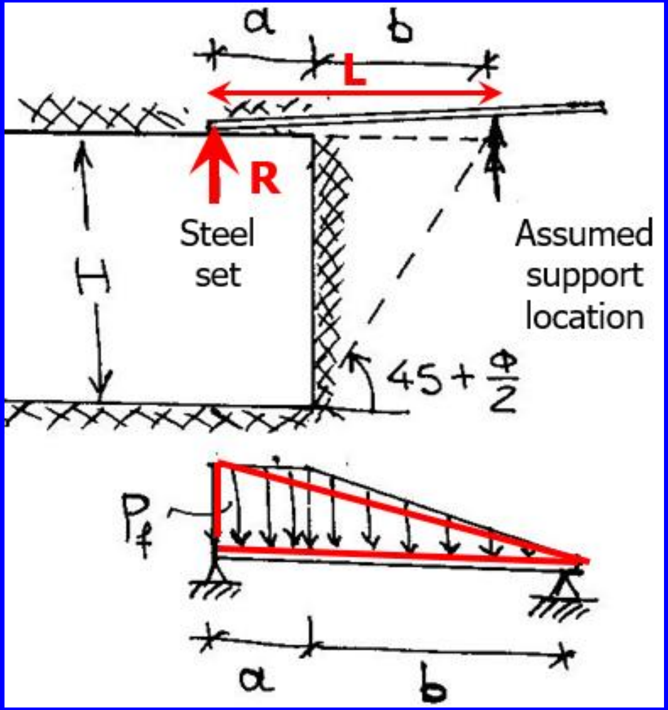
Analysis of Tunnel Face Stability – Support with forepoling

Calculation of reduced deconfinement coefficient (λ) at tunnel face due to the stiffness of the forepoling tubes:

1. Using the convergence-confinement method (for unsupported tunnel), calculate the deconfinement coefficient λ_b at location $x = b = H \tan(45 - \varphi/2)$ (front end of the failing wedge). At this location, there is no effect of the forepoling tubes.
2. Assuming that the forepoling tubes are very stiff, the deconfinement coefficient (λ) at the location where immediate support is applied (about 1m behind the tunnel face, at the location of the last steel set) is equal to λ_b . Thus, $\lambda = \lambda_b$

Analysis of Tunnel Face Stability – Support with forepoling





Very often, modelling of forepoling is performed in a 2D tunnel model, assuming a “reinforced arch” above the tunnel crest (usually with increased E-modulus) to simulate the closely spaced forepoling tubes.

This is not correct, because forepoling tubes undertake forces along their length and NOT as an arch (since there is no contact between the steel tubes, even if they are grouted).

Although an assumed arch at the tunnel crest (with increased E-modulus) also reduces deconfinement, the mechanics of load bearing between the arch and the forepoling tubes are very different (in tunnel plane and normal to tunnel plane, respectively). Thus, such a model cannot be realistic because the axial stiffness of the arch is not correlated to the bending stiffness of the forepoling tubes.

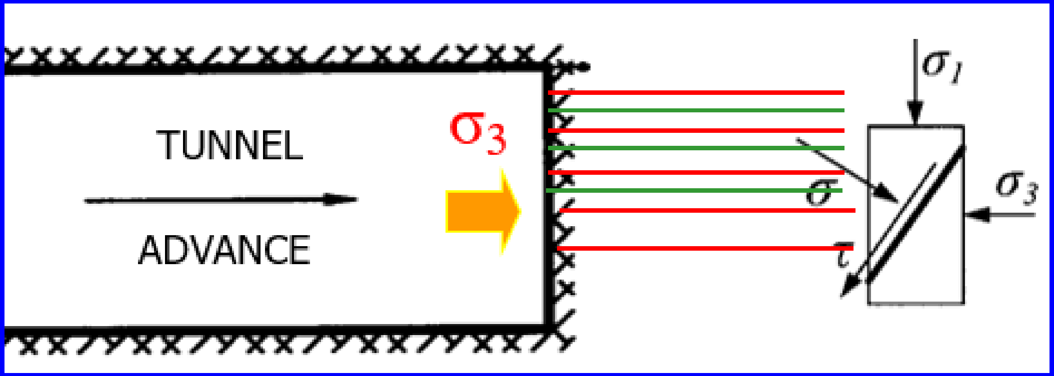
Analysis of Tunnel Face Stability – Support with spiling

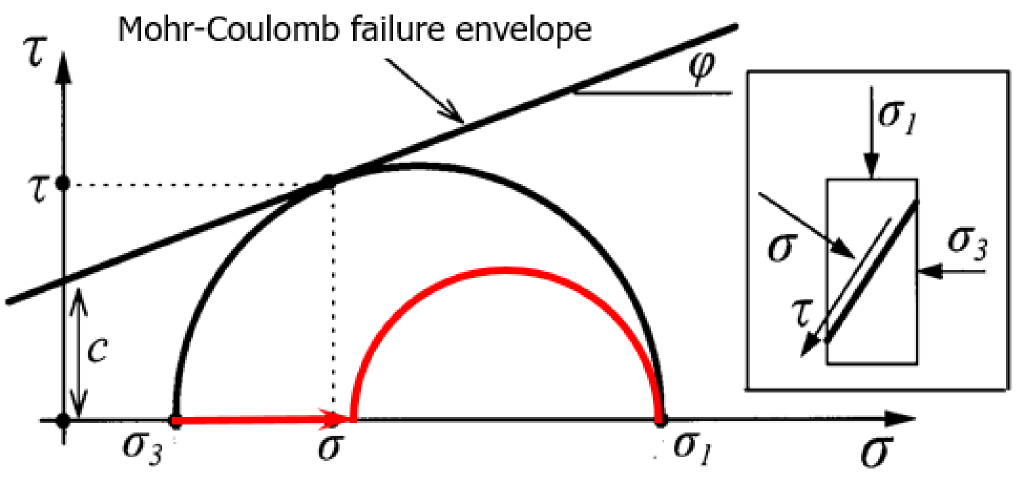
Spiling consists of closely spaced steel bars (20-40mm in diameter) or small diameter tubes (up to 50mm) placed in the upper section of the tunnel. Their objective is to prevent ground raveling in case of cohesionless materials (sandy or gravelly soils, very heavily fractured rockmasses). They are designed empirically (placed as close as required, length 4-6m) and are not part of the structural face support system.

Tunnel Face Stability – Support with Fiber-Glass (FG) nails

FG nails are tensioned as extrusion (inward horizontal movement) of the tunnel face occurs during tunnel advance. Tension in the FG nails results in equal compression of the ground, providing an equivalent horizontal pressure σ_3 .

Tunnel face reinforcement with FG nails reduces very little the deconfinement coefficient (λ) and ground surface settlement, because face extrusion is reduced very little by the FG nails. So, FG nails are not very effective in reducing ground surface settlements, but are very effective in preventing face instability (much more effective than forepoling).





Tunnel Face Stability – Support with Fiber-Glass (FG) nails

Equivalent horizontal pressure σ_3 caused by a grid of FG nails on the tunnel face:

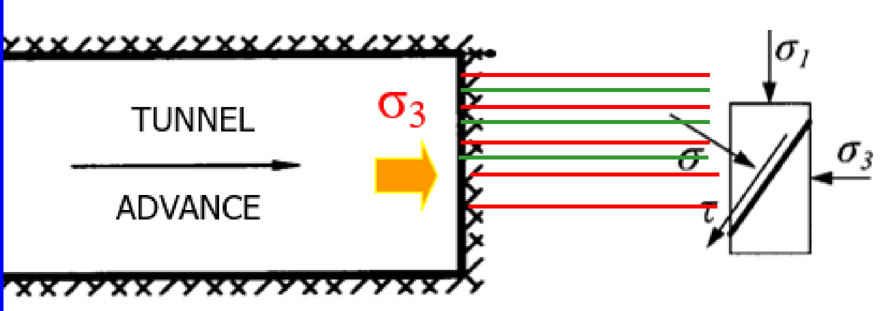
$$\boxed{\sigma_3 = \frac{P}{A} = \frac{n\,F_y}{(FS_F)\,A}}$$

n = number of FG nails

F_y = tensile yield strength of FG nail

FS_F = safety factor of FG nail in tension (usually 1.0 – 1.1)

A = tunnel face area



Ground strength (Mohr-Coulomb) with FG nails: $\sigma_c = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + \sigma_{cm}$

σ_{cm} = rockmass strength

Factor of Safety (FS) of tunnel face supported with FG nails:

$$FS = \frac{\sigma_c}{\sigma_1} = \frac{\sigma_c}{(1-\lambda)\,p_o} \Rightarrow \boxed{FS = FS_o + \frac{1}{(1-\lambda)}\left(\frac{\sigma_3}{p_o}\right)\tan^2\left(45 + \frac{\phi}{2}\right)}$$

where: $FS_o = \dfrac{2}{(1-\lambda)\,N_s}$

FS_o = Factor of safety of the unsupported face

Tunnel Face Stability – Support with Fiber-Glass (FG) nails

Example (same parameters as the example with forepoling):

$\text{GSI}=35,\ \sigma_{ci} = 12\text{ MPa},\ \varphi = 32^o,\ p_o = 75\text{m x } 0.024 = 1.8\text{ MPa}$

Thus: $\sigma_{cm} = 0.95\text{ MPa},\ N_S = 3.8 \Rightarrow \lambda = 0.38 \Rightarrow FS_o = 0.85$

Tunnel face is unstable without support

Required face pressure (σ_3) with FG nails to achieve limiting face stability (FS=1) : σ_3 = 51 kPa

$$\boxed{FS = FS_o + \frac{1}{(1-\lambda)}\left(\frac{\sigma_3}{p_o}\right)\tan^2\left(45+\frac{\phi}{2}\right)}$$

For tunnel face with height H=6m (area A=50 m^2), FG nails with tensile capacity F_y=200 kN (and safety factor FS_F = 1.15), the number of FG nails is: n = 15

$$\boxed{\sigma_3 = \frac{P}{A} = \frac{n\,F_y}{(FS_F)\,A}}$$

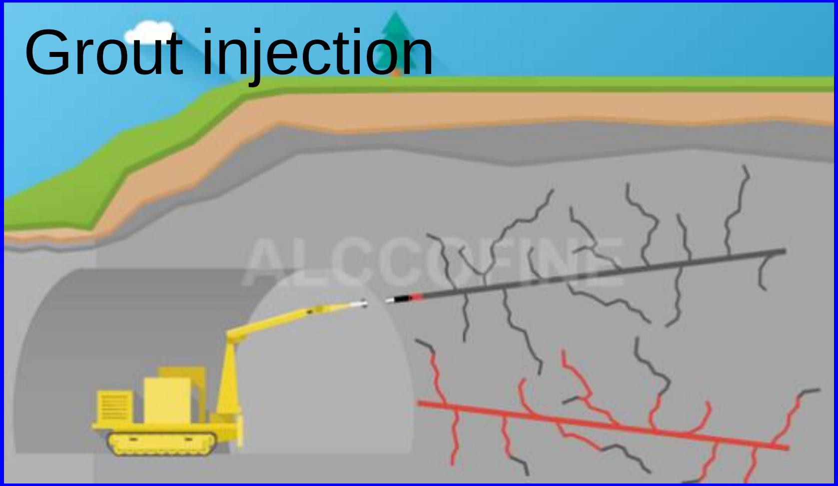
If the depth of the tunnel was 400m (instead of 75m) :

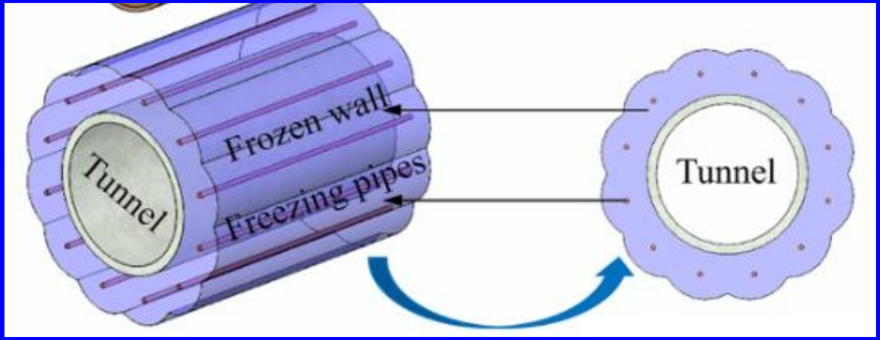
$N_S = 20.3 \Rightarrow \lambda = 0.86 \Rightarrow FS_o = 0.71 \Rightarrow \sigma_3 = 120\text{ kPa}$

The required number of FG nails with above characteristics is: n=35 (reasonable density)

while it is impossible to reach stability with forepoling (required M_{max} is very large)

Tunnel Face Stability – Support with methods increasing ground cohesion like grout injection and ground freezing





Tunnel Face Stability – Support with methods increasing ground cohesion like grout injection and ground freezing

These methods increase ground cohesion by Δc. Friction angle (φ) is not affected.

Ground strength (Mohr-Coulomb) with increased cohesion and $\sigma_3 = 0$:

$$\sigma_c = 2\, c_o \tan\left(45 + \frac{\phi}{2}\right) + 2\, \Delta c \tan\left(45 + \frac{\phi}{2}\right) \Rightarrow \quad \sigma_c = \sigma_{cm} + 2\, \Delta c \tan\left(45 + \frac{\phi}{2}\right)$$

Factor of Safety (FS) of the improved tunnel face:

$$FS = \frac{\sigma_c}{\sigma_1} = \frac{\sigma_c}{(1-\lambda)\, p_o} \Rightarrow \quad \boxed{FS = FS_o + \frac{2}{(1-\lambda)}\left(\frac{\Delta c}{p_o}\right)\tan\left(45 + \frac{\phi}{2}\right)}$$

where: $FS_o = \dfrac{2}{(1-\lambda)\, N_s}$ $\quad$ FS_o = Factor of safety of the unsupported face

Notes:

- Grout injection is only effective in case of voids (porosity, fissures) with opening exceeding a few millimetres. Thus, most types of ground are not injectable (except gravelly soils).
- Ground freezing is very effective (since cohesion increases significantly), but freezing fluids are often not environmentally friendly (leakages are common) – liquid nitrogen is OK but expensive

Tunnel Face Stability – Support with methods increasing ground cohesion like grout injection and ground freezing

Example (same parameters as the example with forepoling):

$\mathrm{GSI}=35$, $\sigma_{ci} = 12$ MPa, $\varphi = 32^o$, $p_o = 75\text{m} \times 0.024 = 1.8$ MPa

thus: $\sigma_{cm} = 0.95$ MPa , $N_S = 3.8 \Rightarrow \lambda = 0.38 \Rightarrow FS_o = 0.85$

Tunnel face is unstable without support

Required increase in cohesion (Δc) to achieve limiting face stability (FS=1) : Δc = 46.4 kPa

$$\boxed{FS = FS_o + \frac{2}{(1-\lambda)}\left(\frac{\Delta c}{p_o}\right)\tan\left(45+\frac{\phi}{2}\right)}$$

If the depth of the tunnel was 400m (instead of 75m) :

$N_S = 20.3 \Rightarrow \lambda = 0.86 \Rightarrow FS_o = 0.71 \Rightarrow$ Δc = 108 kPa

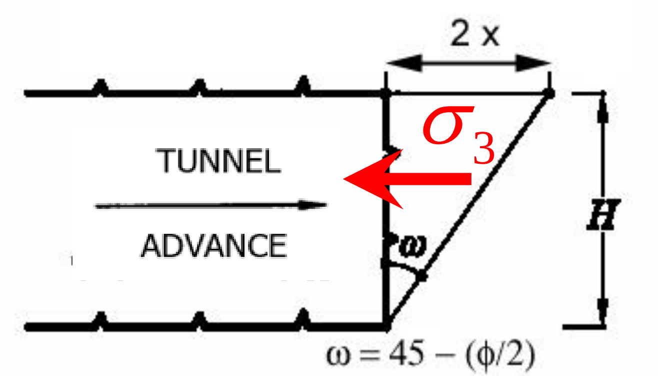
With ground freezing, cohesion can reach values up to 1 MPa. So, ground freezing is a good method (but expensive and slow) to tunnel through very weak ground, especially in cases where ground surface settlements need to be limited (e.g. urban tunnelling, especially in historical cities).

Cement (or even chemical) grouts cannot permeate most ground types, expect gravelly soils or rocks with open fissures (at least a few millimeters wide). In such cases, cohesion increase by about 100-200 kPa is feasible.

Tunnel Face Stability – Support by reduction of hydraulic pressure head

Groundwater flow towards the tunnel face causes seepage forces on the ground with magnitude $f = i\ \gamma_w$ (body force per unit volume), where i = hydraulic gradient and γ_w = unit weight of water. This produces an equivalent outward force $F = f\,V$ (V=volume of the face wedge) and an equivalent outward pressure $\sigma_3 = F / A$ (A=face area):

$$\sigma_3 = i\,\gamma_w\left(\frac{2}{3}2x\right) \qquad i = \frac{h_w}{l} = \frac{h_w}{(2 \div 3)H} \qquad x = \frac{1}{2}H\tan\left(45 - \frac{\phi}{2}\right)$$



Equivalent ground strength (reduced due to σ_3):

$$\sigma_c = \sigma_{cm} - \sigma_3 \tan^2\left(45 + \frac{\varphi}{2}\right)$$

Factor of safety (FS) of the tunnel face:

$$FS = \frac{\sigma_c}{\sigma_1} = \frac{\sigma_c}{(1-\lambda)\,p_o} \Rightarrow \boxed{FS = FS_o - \left[\frac{2}{3(1-\lambda)}\left(\frac{\gamma_w H}{p_o}\right)\tan\left(45 + \frac{\phi}{2}\right)\right] i}$$

where: $FS_o = \dfrac{2}{(1-\lambda)N_s}$ (FS_o = factor of safety without water flow)

Tunnel Face Stability – Support by reduction of hydraulic pressure head

Example (same parameters as the example with forepoling):

GSI=35, σ_{ci} = 12 MPa, $\varphi = 32^o$, p_o = 50m x 0.024 = 1.2 MPa

thus: σ_{cm} = 0.95 MPa , N_S = 2.5 $\Rightarrow$ λ = 0.295 $\Rightarrow$ FS_o = 1.135

Face is stable without water flow

$$\boxed{FS = FS_o - \left[\frac{2}{3(1-\lambda)}\left(\frac{\gamma_w H}{p_o}\right)\tan\left(45+\frac{\phi}{2}\right)\right] i}$$

Stability with water flow:

h_w = 41.5 m (piezometric head at tunnel face)
l = 15m (seepage length) $\Rightarrow$ i = h_w / l = 2.765
γ_w = 10 kN/m^3 (unit weight of water), H = 6 m (height of tunnel face)

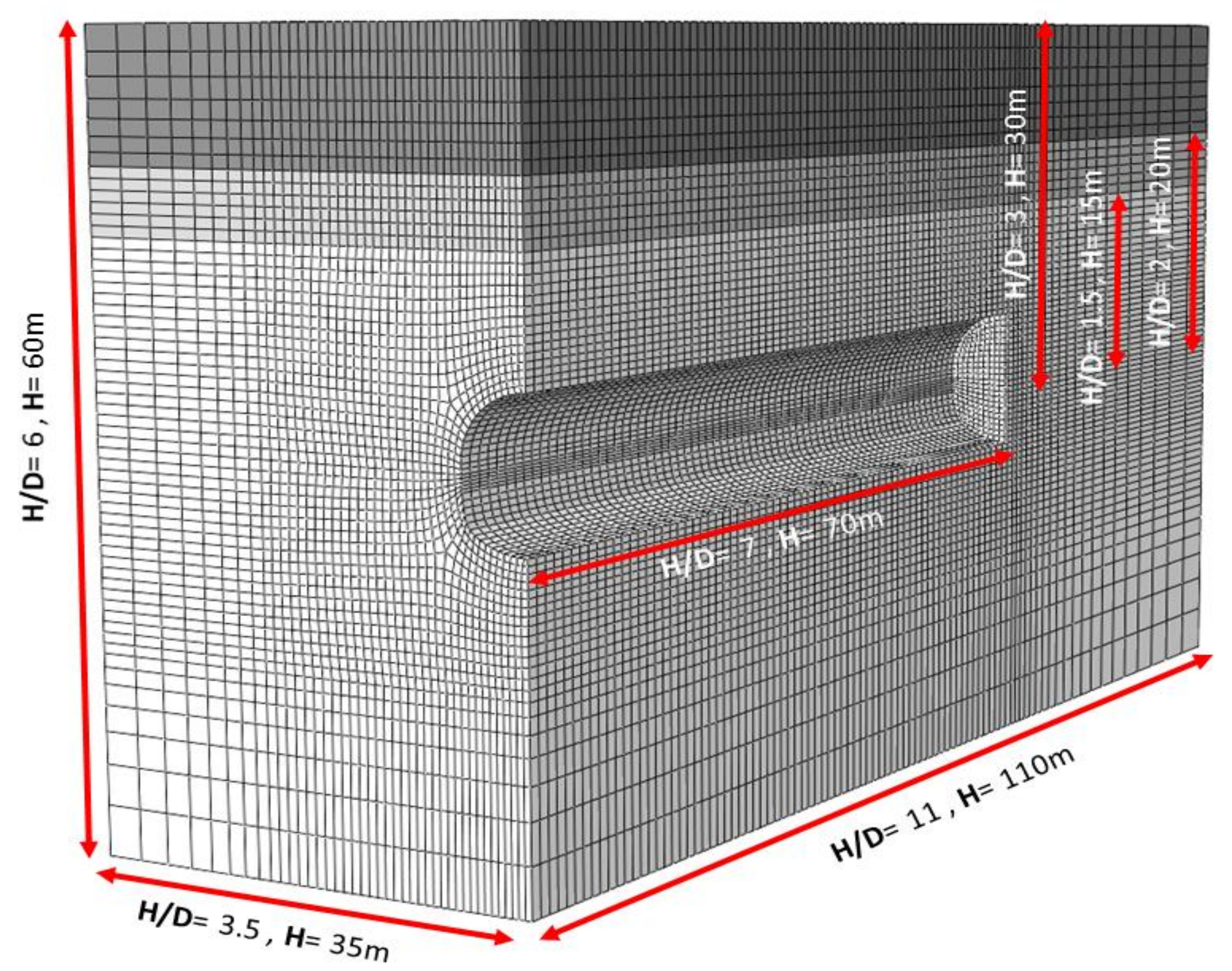
Factor of safety of the tunnel face with water flow:

FS = 1.135 – 0.085 i = 1.135 – 0.235 = 0.90 (face is unstable)

To achieve limiting face stability (FS=1), the hydraulic gradient should be: i = 1.59, i.e., the piezometric head at tunnel face should be:

$h_w = i\, l$ = 1.59 x 15 = 23.9 m (reduction by 42%)

Analysis of tunnel face stability (PhD thesis, D. Georgiou 2021)



3D finite element analyses with a wide range of ground and depth parameters to calculate face stability

Analysis of tunnel face stability (PhD thesis, D. Georgiou 2021)

$$\sigma_{cm} = 2c\,tan\left(45° + {}^{\varphi}/_{2}\right)$$

$$\sigma_{cm} = 0.02\,\sigma_{ci}\,exp\left(\frac{GSI}{25.5}\right)$$

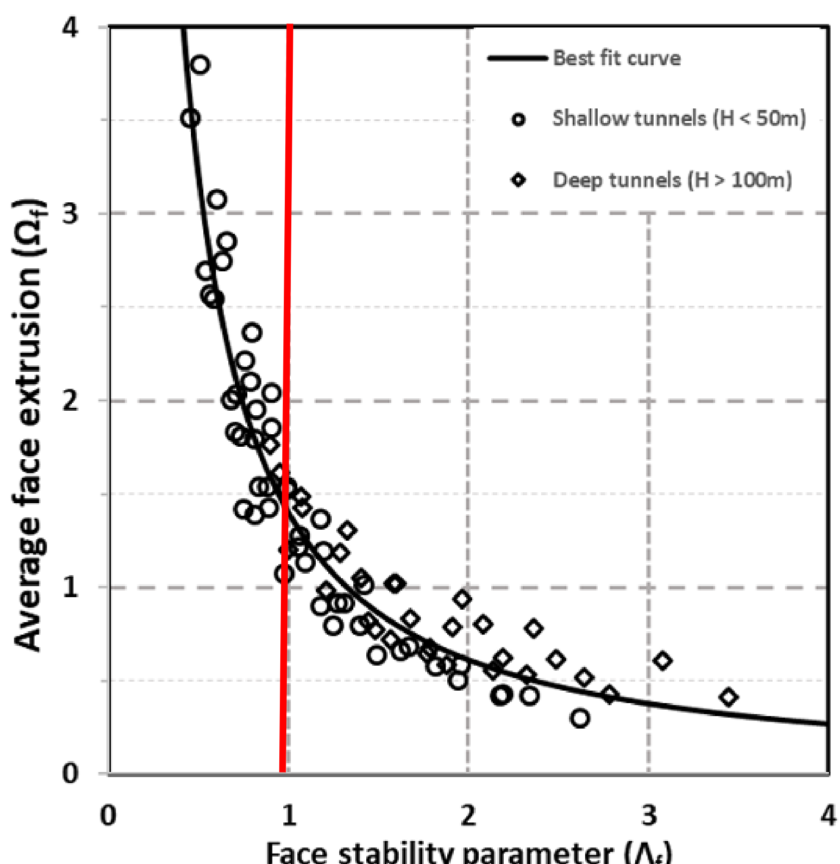
$$\Omega_f = \left(\frac{U_h}{D}\right)\left(\frac{E}{p_o}\right)$$

Dimensionless average face extrusion

U_h = average face extrusion

Dimensionless face stability factor:

$$\Lambda_f = 3.8\left(\frac{\sigma_{cm}}{\gamma H\sqrt{1+(2/3)K_o}}\right)\left(\frac{H}{D}\right)^{0.35}$$



H = tunnel depth

σ_{cm} = ground strength

D = tunnel width

K_o = horizontal stress factor

Results of numerical analyses:

Face is unstable if $\Lambda_f < 1$

Λ_f = Factor of safety against face instability