INFRASTRUTTURE VIARIE IN SOTTERRANEO

ROCK SUPPORT

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Effect of Excavation on Rock Mass

When considering the principles of rock mass stabilization, there are two aspects of rock excavation that must be considered:

- ✧ The first is that 'one cannot prevent all displacements at the excavation boundary'.
- ✧ The second is that ' mistakes in excavation design can be a major problem'.

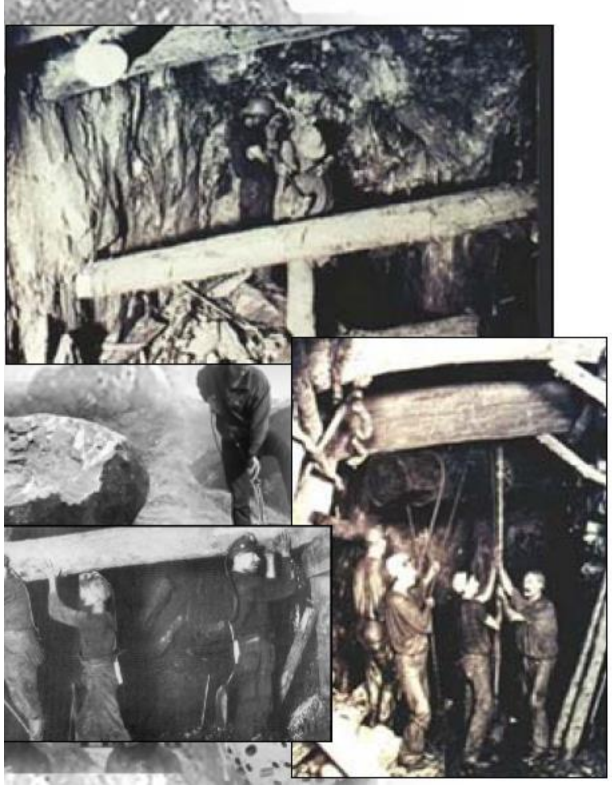
Boys working during World War I coal boom.

Photograph from U. S. Bureau of Mines

A TINY GUARDIAN OF THE MINER'S WELFARE

As susceptible as men are to the overwhelming effects of mine gases, the canary bird is much more so. The result is that in many disasters the birds are made the outposts of the invading army of restoration. They are overcome long before man can detect the presence of the gas and therefore warn the men of the dangers ahead.

Historic Ground Control Measures



Photograph from U. S. Bureau of Mines

LINING A MINE WALL WITH ARTIFICIAL ROCK

One of the frequent causes of mine cave-ins is the weathering of the slate of the roof and side walls. It gradually crumbles or scales off and suffers a consequent weakening, which may finally bring disaster. The cement gun covers the slate with a thin plaster, which effectually shuts out the air and leaves it as unexposed to deterioration as it was during the countless ages before the coal was removed.

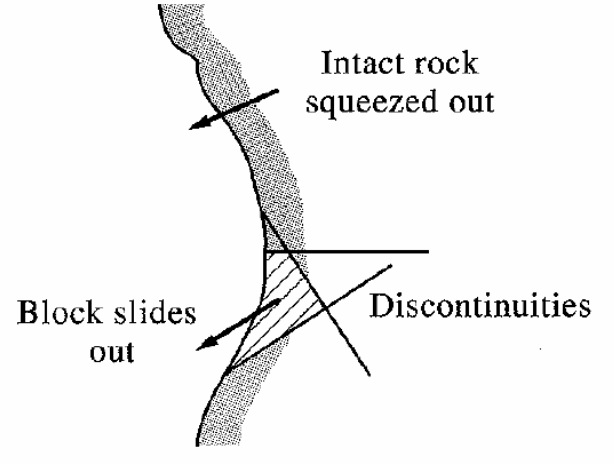
Effect of Excavation on Rock Mass

In order to understand the displacements and avoid problems, we must consider the three primary effects of excavation and then decide on the ramifications for stabilizing excavations of all kinds.

The three primary effects of excavations are:

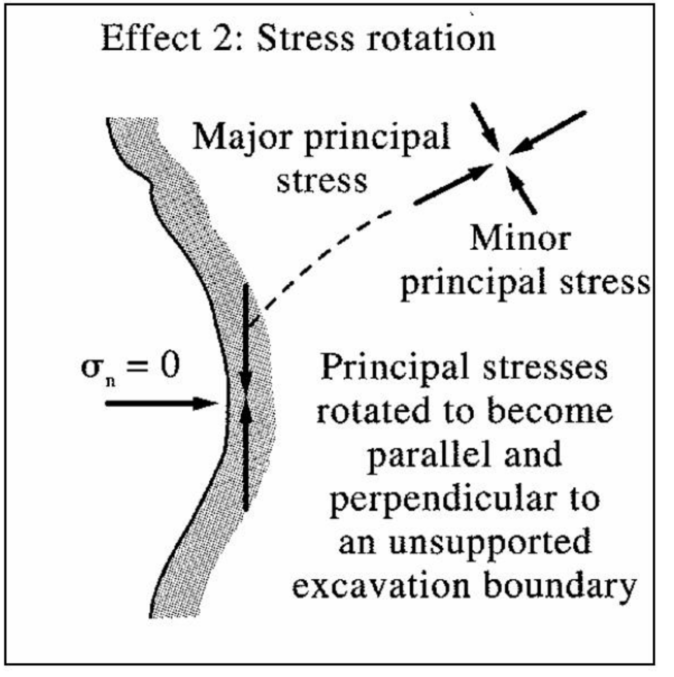
1) Displacements occur because stressed rock has been removed, allowing the remaining rock to move (due to unloading).

Effect 1: Displacements and rock failure



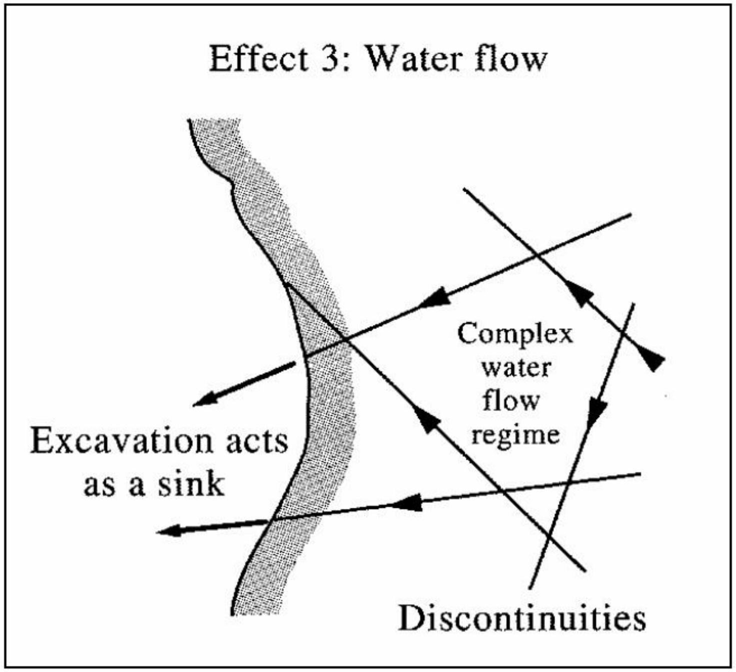
Effect of Excavation on Rock Mass

2) There are no normal or shear stresses on an unsupported excavation surface and hence the excavation boundary must be a principal stress plane with one of the principal stresses (of magnitude zero) being normal to the surface. Generally, this will involve a major perturbation of the pre-existing stress field, both in the principal stress magnitudes and their orientations.



Effect of Excavation on Rock Mass

3) At the boundary of an excavation open to the atmosphere, any previous fluid pressure existing in the rock mass will be reduced to zero (or more strictly, to atmospheric pressure). This causes the excavation to act as a 'sink', and any fluid within the rock mass will tend to flow into the excavation.

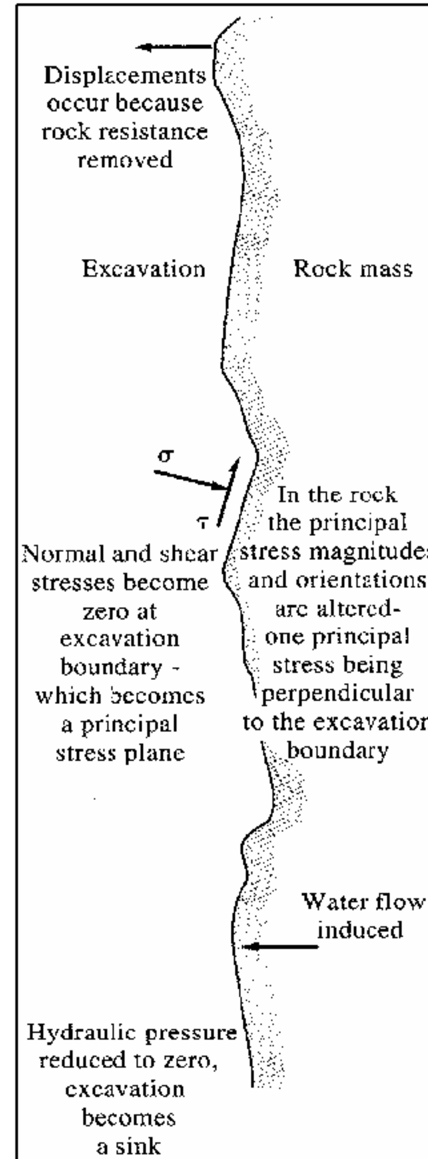


Effect of Excavation

Displacements: the engineering objective dictates the significance of any rock displacement and its maximum tolerable magnitude. It is important to know whether the displacements are associated with entire rock blocks moving into the excavation, whether the rock mass is deforming as a whole, or whether failure is occurring in the rock.

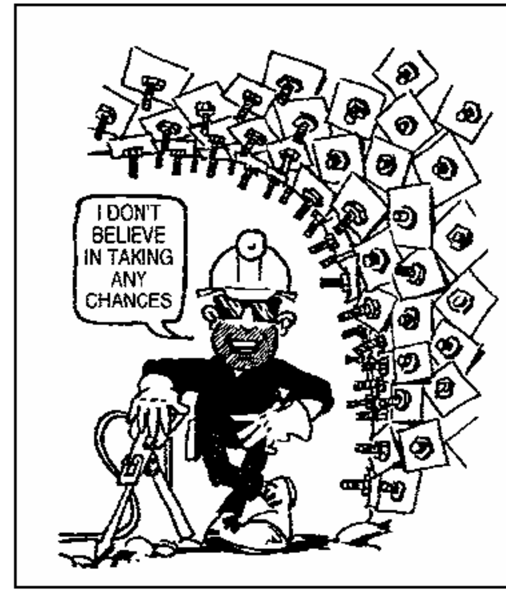
Stress Field: the significance of stress field disturbance is that rock is more likely to fail, owing to the increased magnitude of the deviatoric stresses.

Water Flow: increased water flow is significant because there will be higher differential heads within the rock mass which tend to push rock blocks into the excavation, with the attendant possibility of increased weathering and time dependent deterioration.



Effect of Excavation on Rock Mass

These three primary effects, and the optimal way in which the rock engineering strategy is developed to account for them, have one thing in common: we should not blindly attempt to maintain the original conditions (e.g. by installing massive support or reinforcement and hydraulically sealing the entire excavation).



As the displacements occur, engineering judgement may determine that they can be allowed to develop fully, or be controlled later.

The Stabilization Strategy

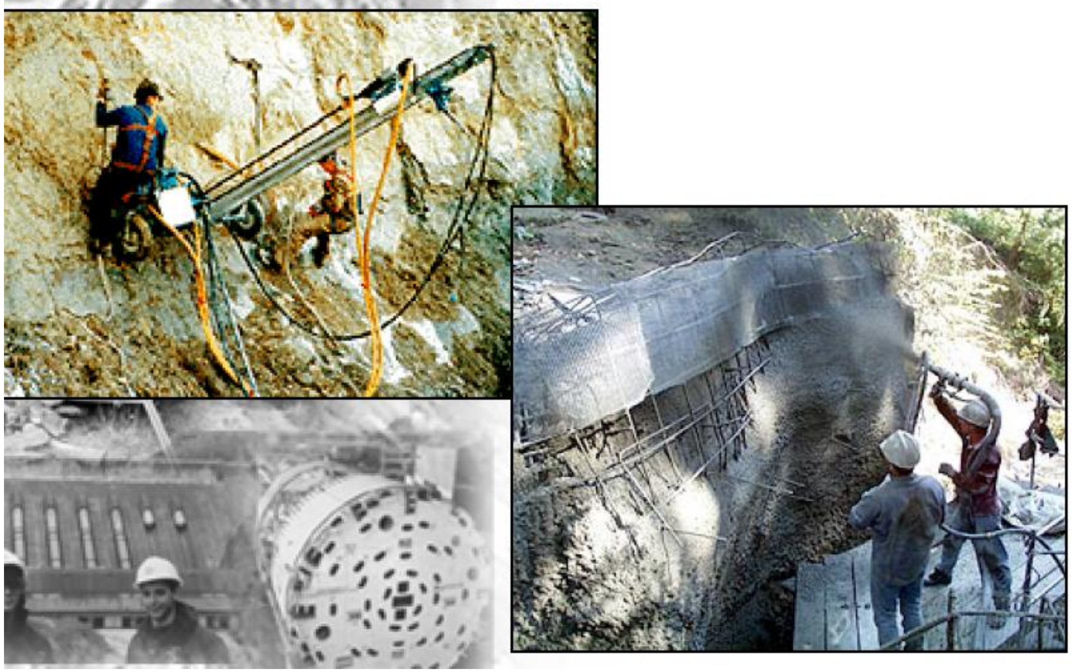
If failure around an excavation, whether at surface or underground, is due to blocks of rock moving into the excavation, two approach philosophies can be considered for stabilization:

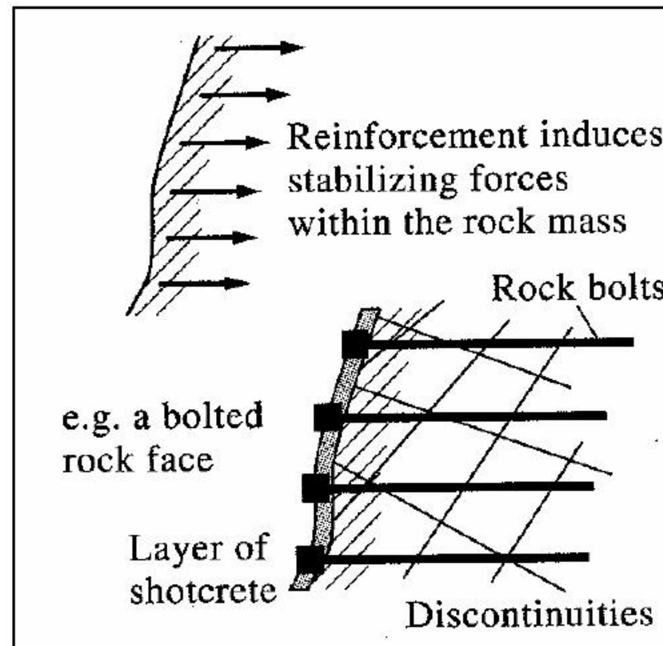
- the block displacements are occurring because the rock mass is a discontinuum, and hence the rock is reinforced so that it behaves like a continuum; or
- direct support elements are introduced into the excavation in order to maintain block displacements at tolerable levels.

The first option is known as rock reinforcement; the second is known as rock support (or retainment).

The Stabilization Strategy

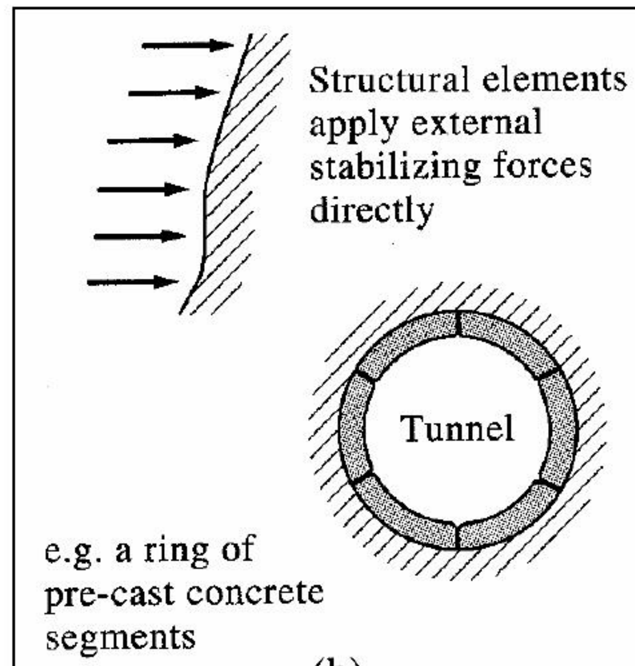
In the case of reinforcement, steel cables or bolts grouted within boreholes are used to minimize displacements occurring along the discontinuities – *so that the rock supports itself*. In conjunction with bolting, sprayed concrete (shotcrete) is used to protect the surface and inhibit minor block movements.



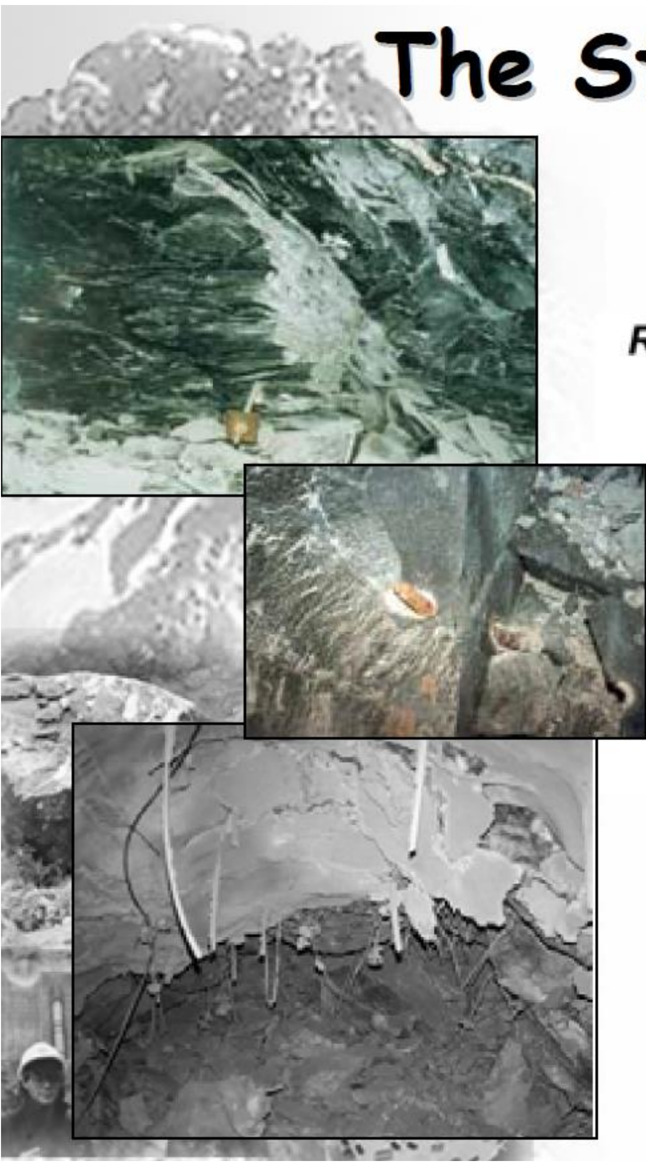


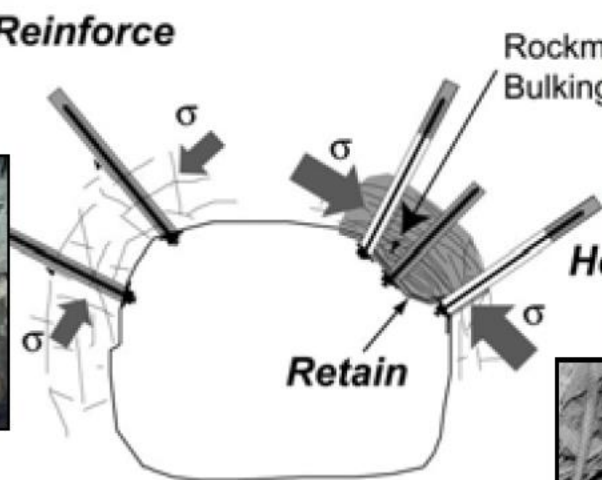
The Stabilization Strategy

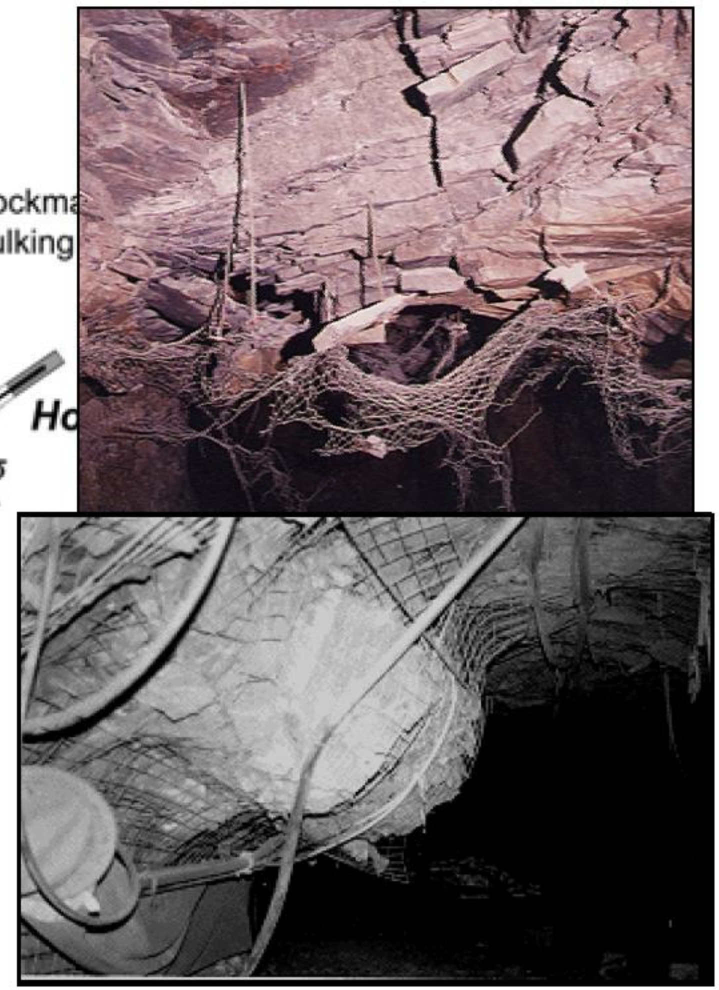
In the case of support/retainment, structural elements – such as steel arches or concrete rings – are introduced to inhibit rock displacements at the boundary of the excavation. These elements, which are external to the rock mass, provide load bearing capability, with the result that – *the rock is supported*.



The Stabilization Strategy

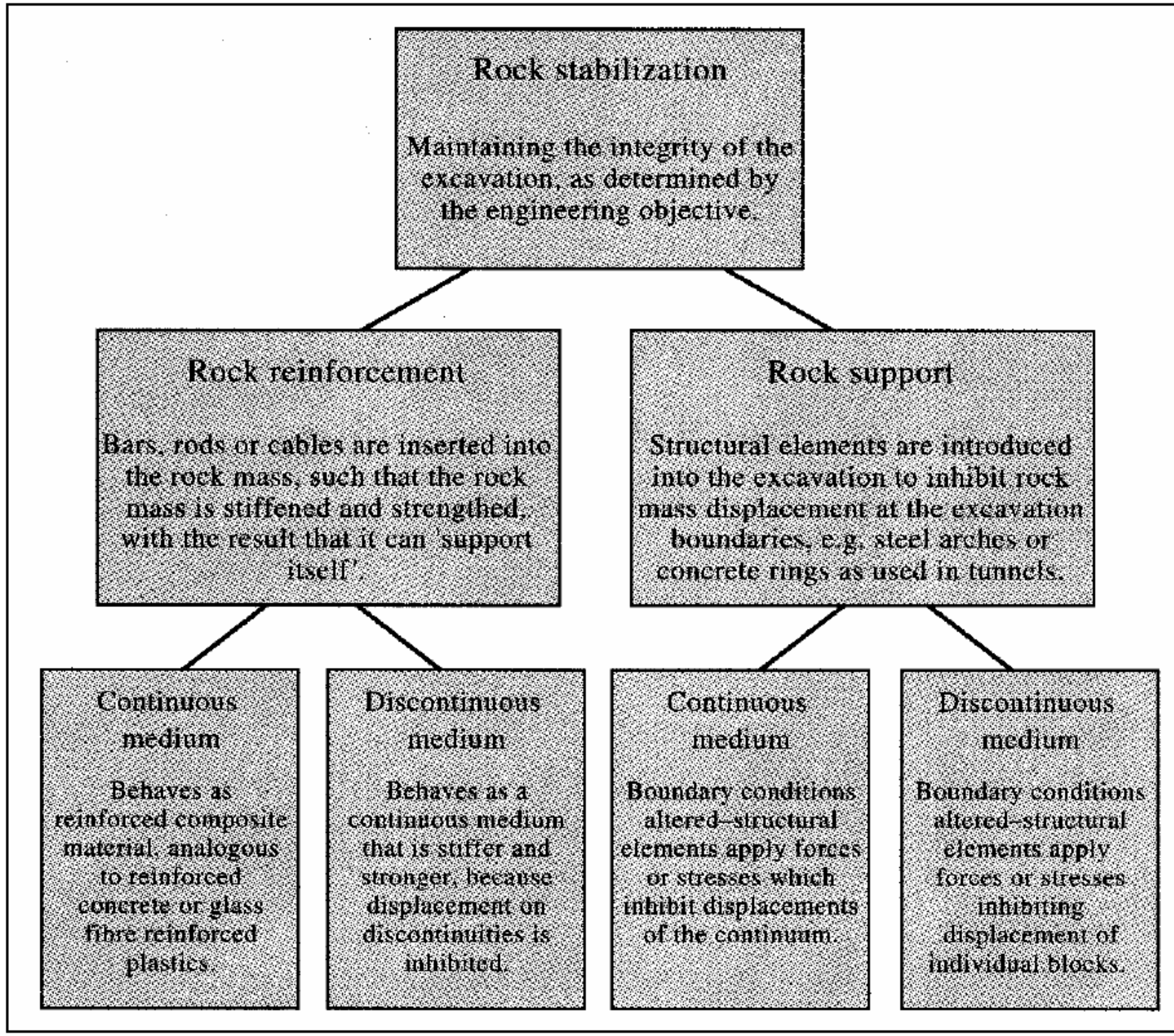




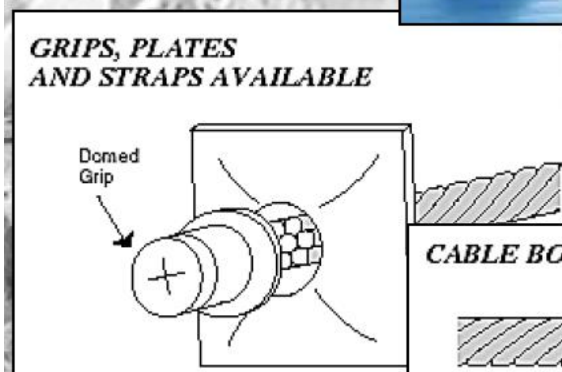


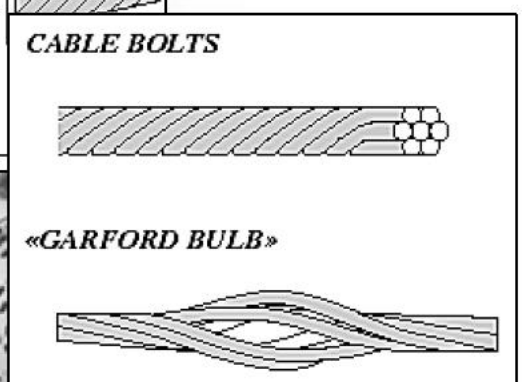
The Stabilization Strategy

… note that with rock reinforcement the engineering elements are inserted within the rock mass and with support they are applied within the excavation.

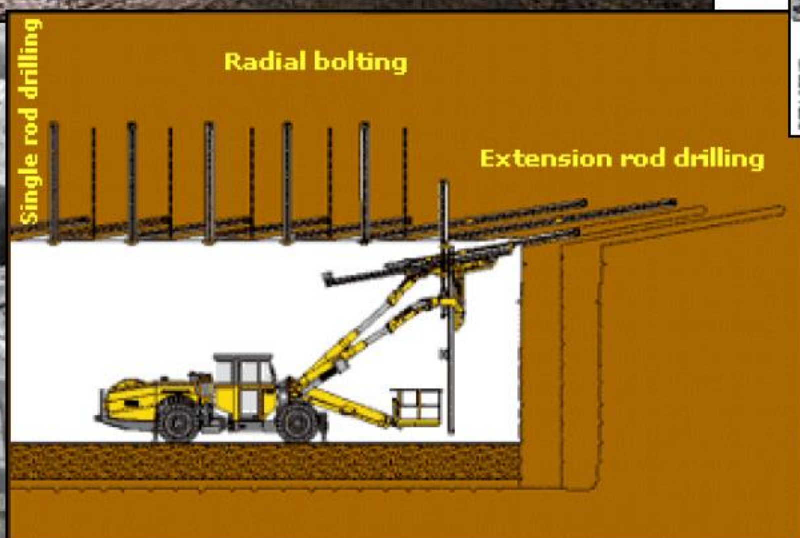


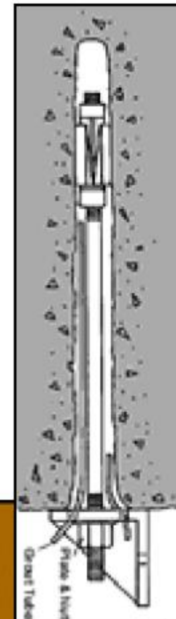
Rock Reinforcement

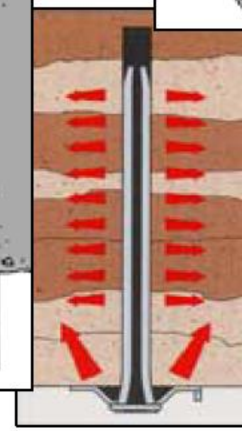


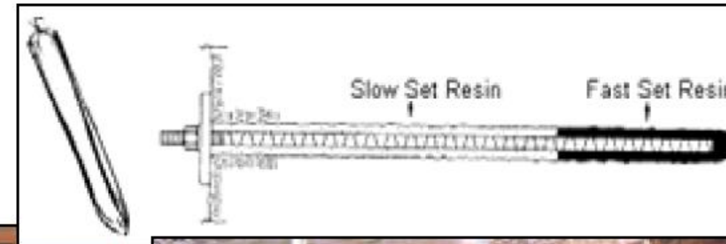


Rock Reinforcement Installation



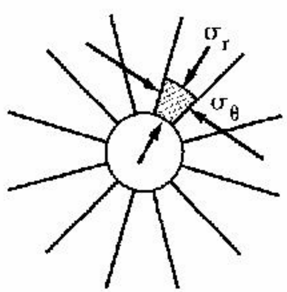






Rock Reinforcement in Continuous Rock

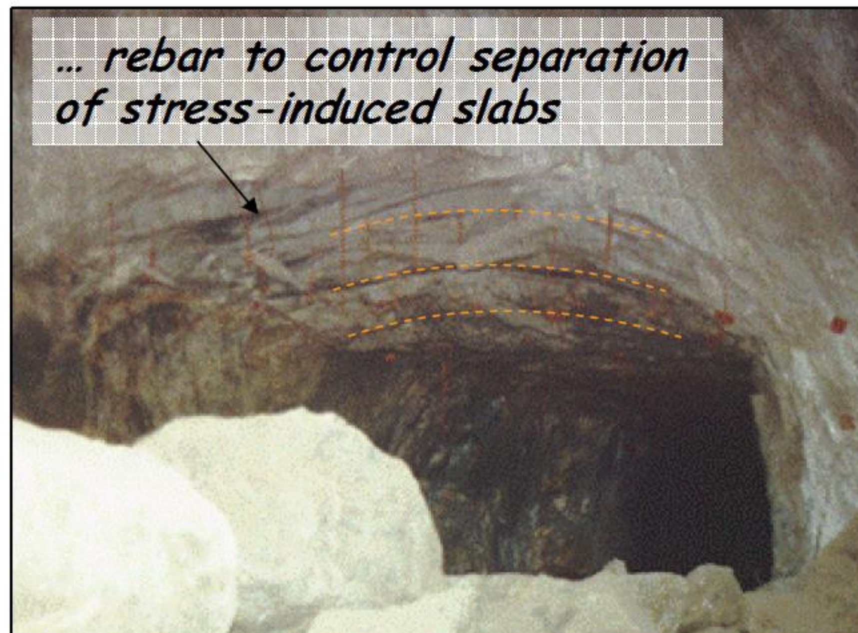
It may be thought that the use of rock reinforcement is only of use in discontinuous rock masses in order to prevent discrete block displacements. However, the use of rock reinforcement in a continuous medium can also be of benefit, especially with respect to brittle failure processes, because of the added confinement, controlling of displacements and reduction in rock mass bulking/dilation.



Sketch of radially reinforced annulus

$$\sigma_r = A'E'\nu\sigma_\theta$$

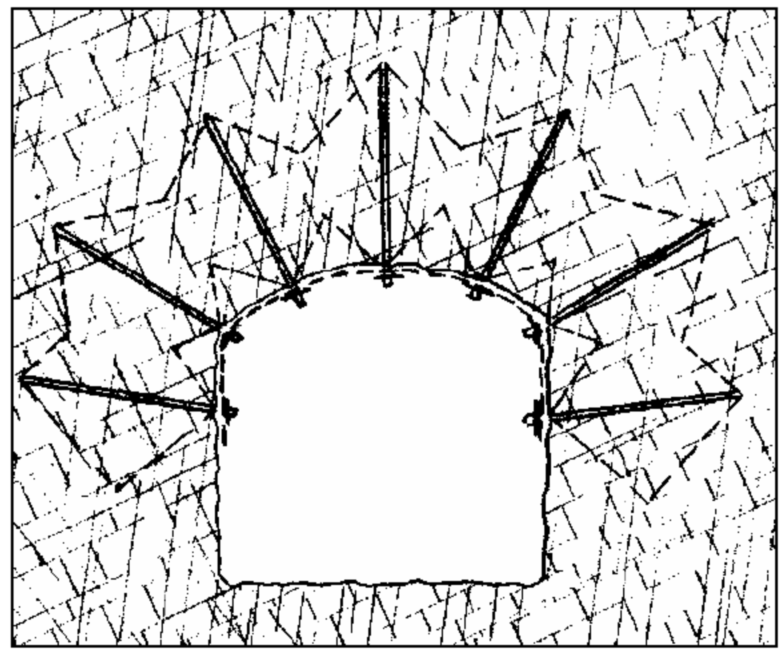
where A' and E' are the ratios of the cross-sectional areas and the Young's moduli of the reinforcing element to that of the rock being reinforced, respectively,
ν is Poisson's ratio for the rock, and
σ_θ is the tangential stress.



Rock Reinforcement in Discontinuous Rock

The mode of action of the reinforcement in a discontinuous medium is somewhat different, because not only are we considering improvement of the rock structure properties, but also the avoidance of large displacements of complete blocks.

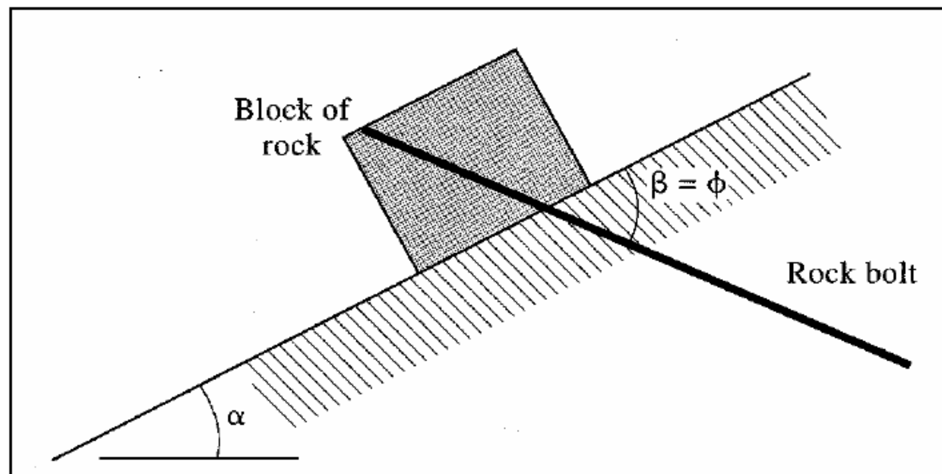
Two of the most important factors are whether the blocks are free to move, given the geometry of the rock mass and excavation (i.e. kinematic feasibility), and the character (quantity, length and orientation) of the reinforcement.



Rock Reinforcement in Discontinuous Rock

The simplest case of reinforcing a discontinuous material is that of a single block on a rock surface reinforced by a tension anchor. The tension anchor should be installed such that the block and the rock beneath act as a continuum, and block movement is inhibited.

Without the bolt, basic mechanics indicates that the block will slide if the slope angle exceeds the friction angle of the rock surface (for a cohesionless interface). This is the first criterion for indicating the potential for failure.

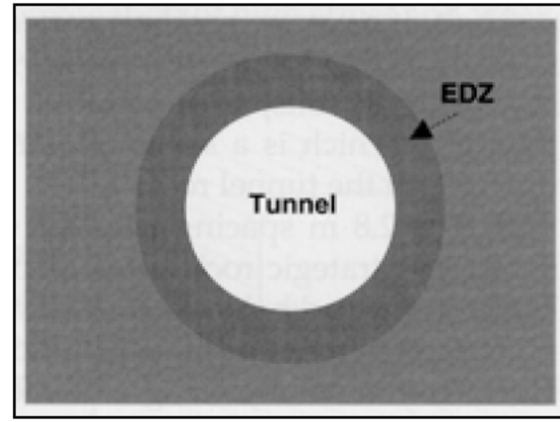


Rock Reinforcement in Discontinuous Rock

Considering now the length and diameter of the bolt, these have to ensure that the strength of the bonds across the anchor-grout and grout-rock interfaces are capable of sustaining the necessary tension in the anchor, which in turn will depend on the fracturing of the rock mass. Furthermore, the anchor diameter may also be determined on the basis of the tensile strength of the anchor material.

Example #1: Rock Reinforcement

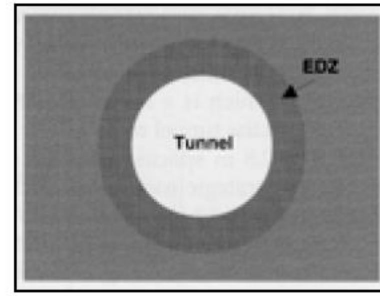
Q. A circular tunnel is being excavated in a blocky rock mass by drilling and blasting. There is an Excavation Disturbed Zone (EDZ) around the excavated tunnel (defined on the basis of a blast-disturbed zone where there are loosened blocks which can fall into the tunnel under the action of gravity) which extends 0.75 m into the rock from the excavation surface.



What reinforcement pressure is required at the crown to stabilize the loose blocks of the EDZ given that the unit weight of the rock, γ, is 25 kN/m^3.

Example #1

Q. What reinforcement pressure is required at the crown to stabilize the loose blocks of the 0.75 m thick EDZ (γ = 25 kN/m^3).



A. The reinforcement pressure, *p*, can be approximated as *W/A*, where *W* is the weight of the loose blocks and *A* is the surface area of the tunnel being considered.

Taking the EDZ volume, *V*, above a 1 m^2 area of tunnel roof, the weight of the EDZ is:

$$W = \gamma V = 25 \times (0.75 \times 1 \times 1) = 18.75\ kN$$

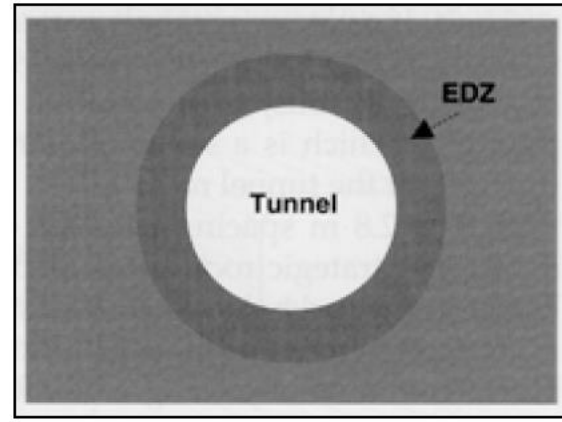
The area considered is 1 x 1 = 1 m^2, therefore the support pressure, p, is:

$$p = \frac{W}{A} = \frac{18.75\ kN}{1\ m^2} = 18.75\ kPa$$

Example #2: Rock Reinforcement

Q. Same problem:

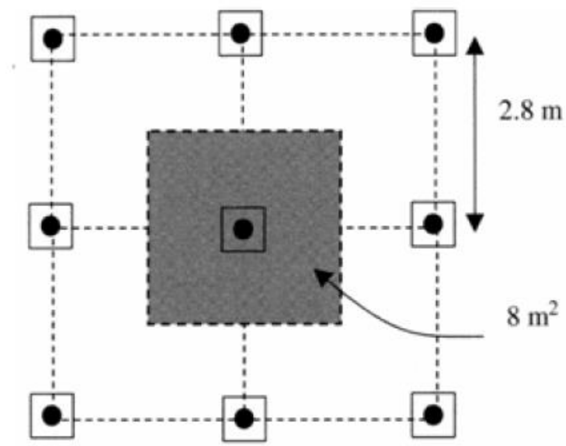
If this EDZ is to be stabilized by the use of rockbolts inserted into the roof as a supporting method, and the working capacity of each bolt, *T*, is 150 kN, what area of the roof will each bolt support.



A. If a bolt can sustain a load of 150 kN and the support pressure, *p*, is 18.75 kPa, then:

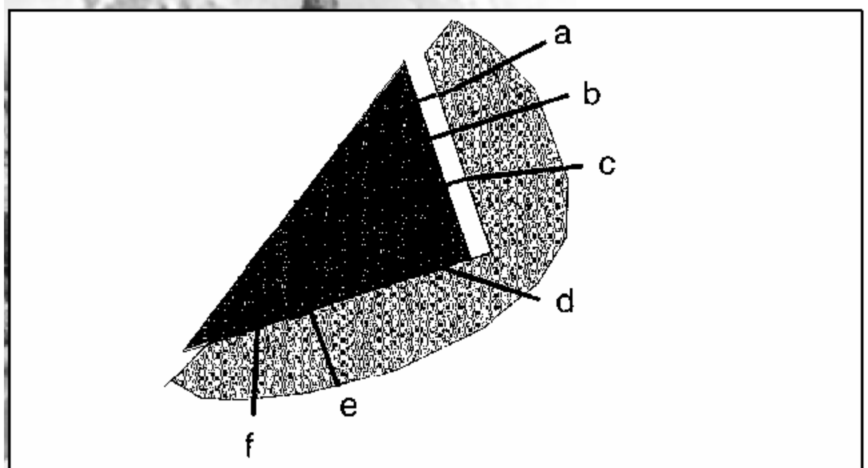
$$p = \frac{150\ kN}{18.75\ kN/m^2} = 8\ m^2$$

of roof per rockbolt.

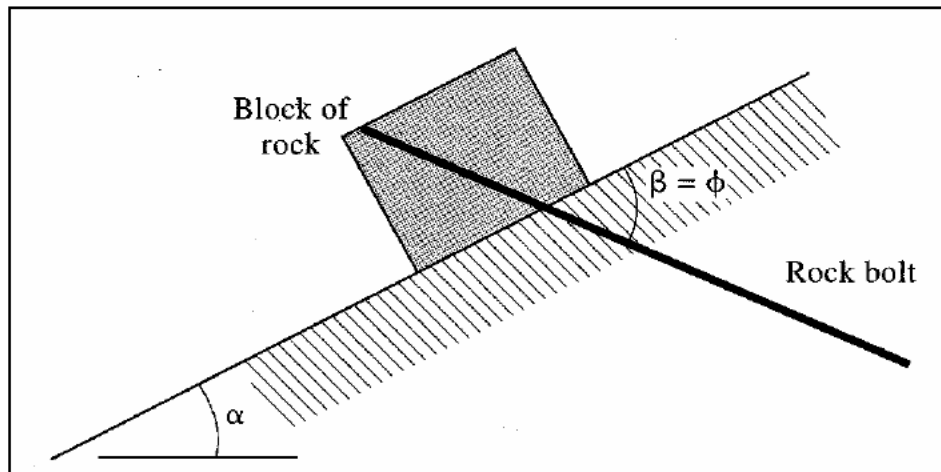


Rock Reinforcement in Discontinuous Rock

With respect to the bolt orientation and tension, it is not always obvious at what angle the anchor should be orientated for optimal effect. If we regard the optimal orientation as that which enables the anchor tension to be a minimum, then the angle between the anchor and the slope surface is equal to the friction angle between the block and the slope.



a: tension and shear b: tension only c: tension and shear
d: shear and tension e: shear only f: shear and compression



Active and Passive Reinforcement

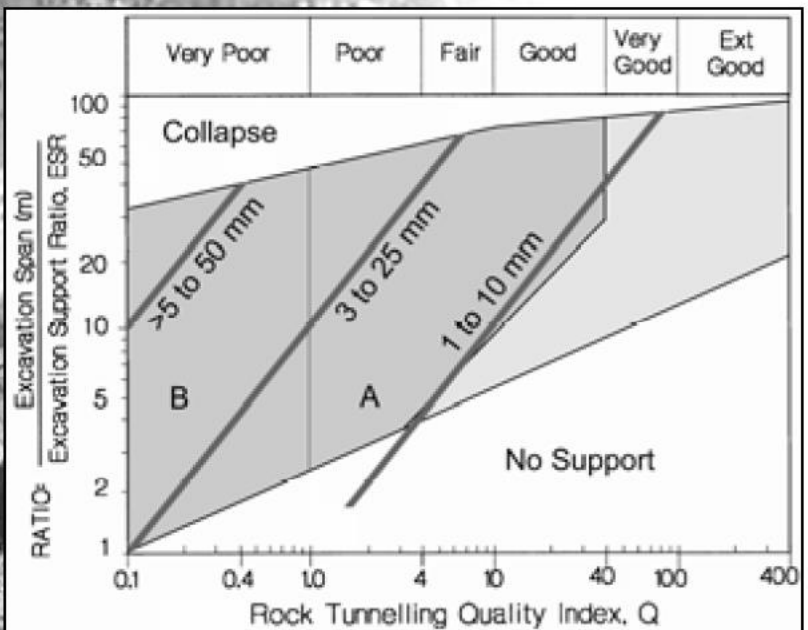
Rock reinforcement may also be classified as active or passive:

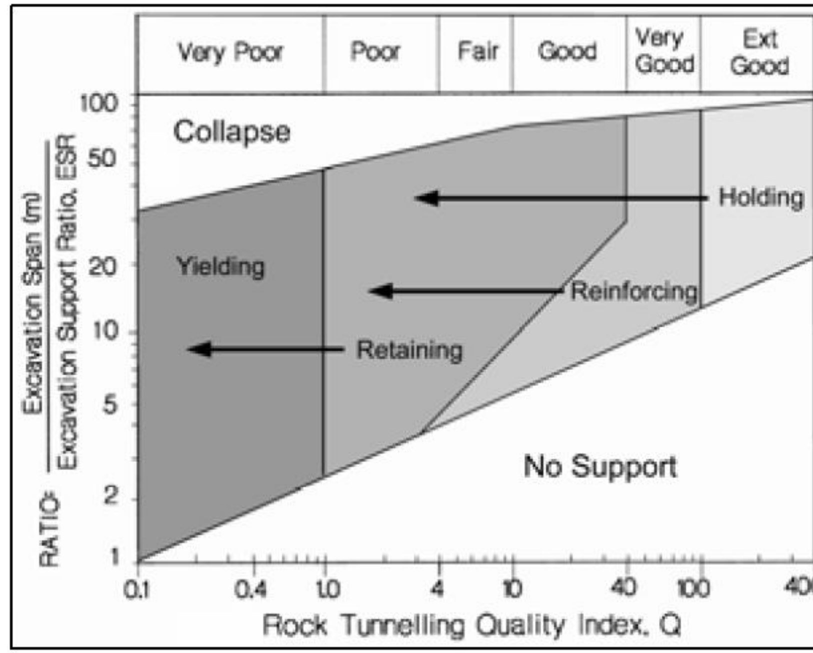
Active support is installed with a predetermined load to the rock surface (e.g. tensioned cables or bolts). Active support is usually required when it is necessary to support the gravity loads imposed by individual rock blocks.

Passive support is not installed with an applied load, but rather develops its loads as the rock mass deforms (e.g. grouted bars, friction bolts, shotcrete, wire mesh). Passive support therefore requires rock displacement to function.

Rock Reinforcement in Yielding Rock

With ground control problems in brittle rock, support selection must be thought of with respect to the anticipated deformation NOT load.





... deformation limits observed from well supported excavations superimposed on stability chart (after Kaiser et al. 1996).

Rock Reinforcement in Yielding Rock

Basic Requirements:

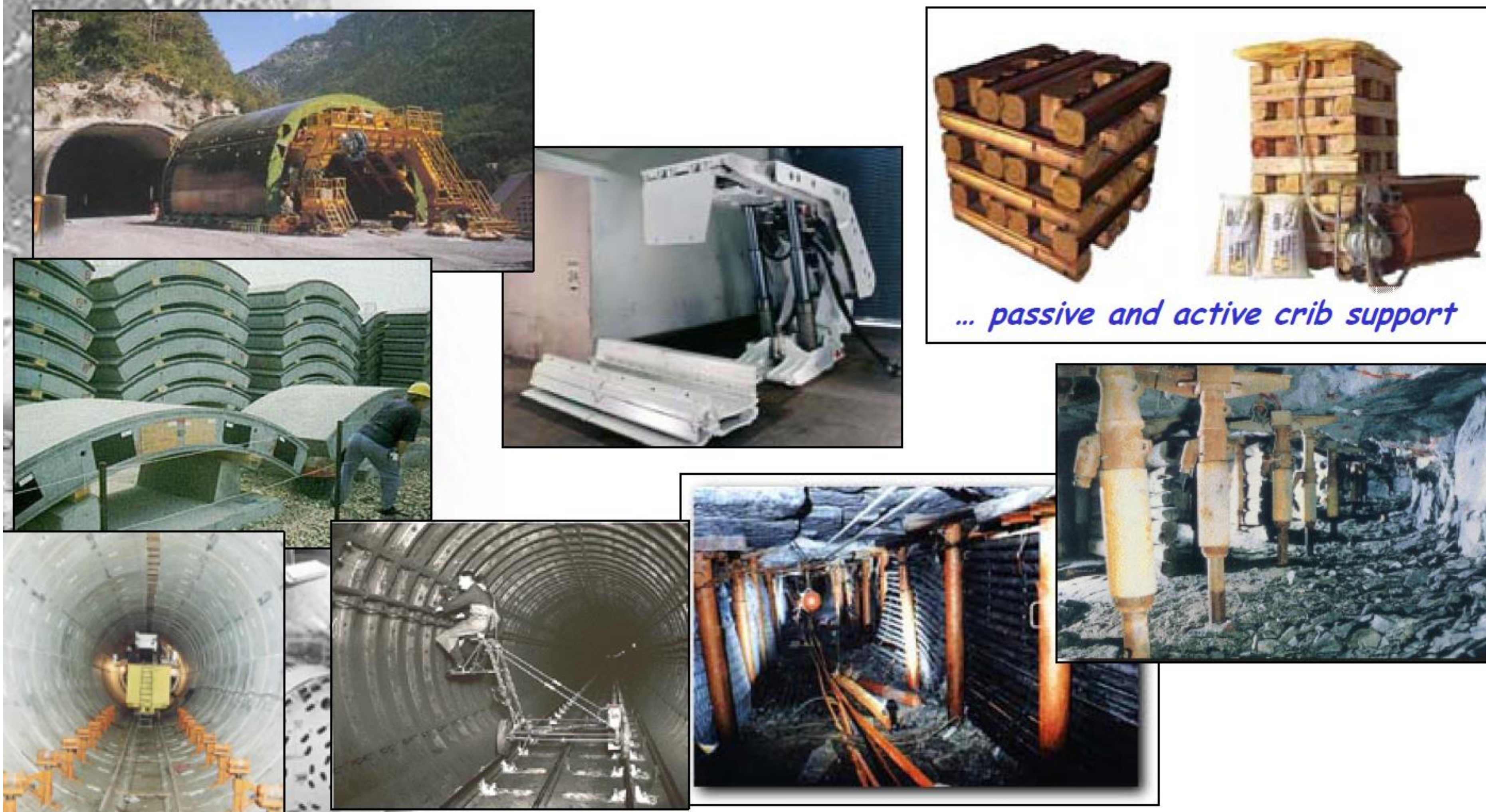
- *In situ* stress
- Geometry
- Rock mass strength or failure criteria

⇨ to predict depth of failure

- Rock mass bulking factor

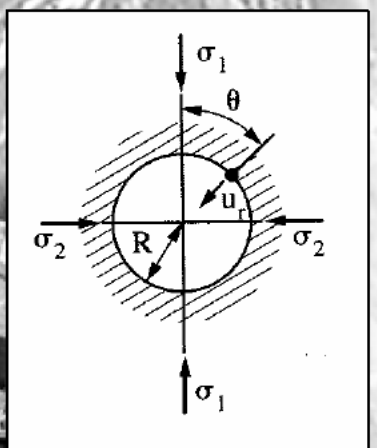
⇨ to determine convergence or rock mass straining

Rock Support



Rock Support in Yielding Rock

Consider the stresses and displacements induced by excavating in a continuous, homogeneous, isotropic, linear elastic rock mass. The radial boundary displacements around a circular tunnel assuming plane strain conditions are:



$$u_r = (R/E)[\sigma_1 + \sigma_2 + 2(1 - \nu^2)(\sigma_1 - \sigma_2)\cos 2\theta - \nu\sigma_3]$$

where R is the radius of the opening,
σ_1 and σ_2 are the far-field in-plane principal stresses,
σ_3 is the far-field anti-plane stress,
θ is indicated in the margin sketch, and
E and ν are the elastic constants.

The rock stabilization strategy can be based on the need to restrict the displacements as governed by the engineering objective.

Rock Support in Yielding Rock

The ground response curve is a graph of the support pressure required to maintain equilibrium of the boundary for a given displacement value. Where the elastic ground response curve intersects the boundary displacement axis, the u_r value, represents the total elastic deformation of the boundary of the excavation when support pressure is not provided. Typically only values less than 0.1% of the radius would be acceptable for most rock engineering projects.

$$u_r = (R/E)[\sigma_1 + \sigma_2 + 2(1 - \nu^2)(\sigma_1 - \sigma_2)\cos 2\theta - \nu\sigma_3]$$

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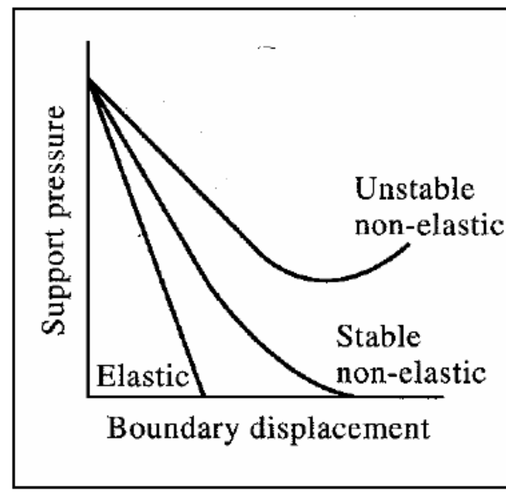
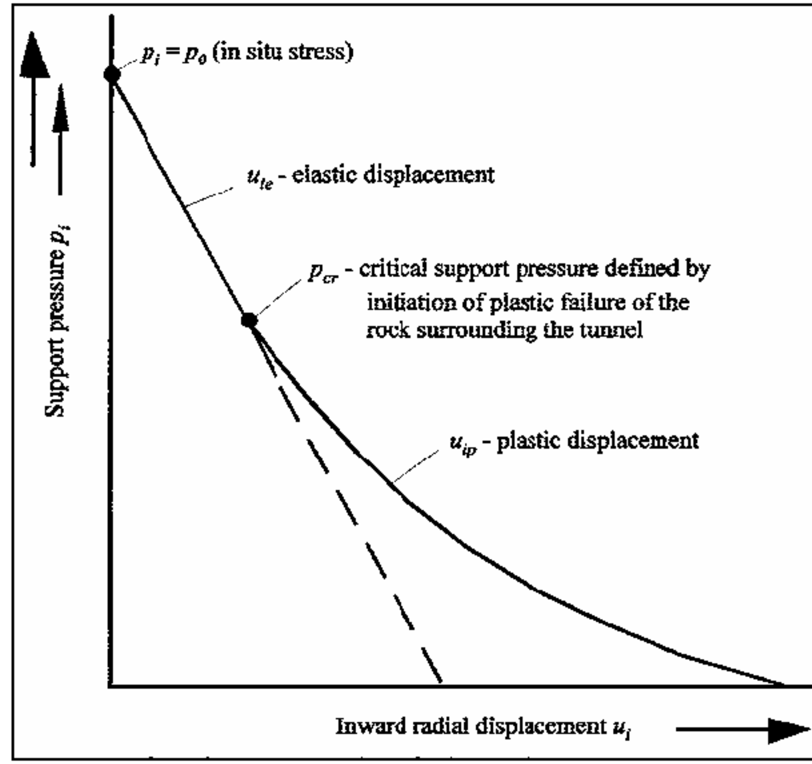
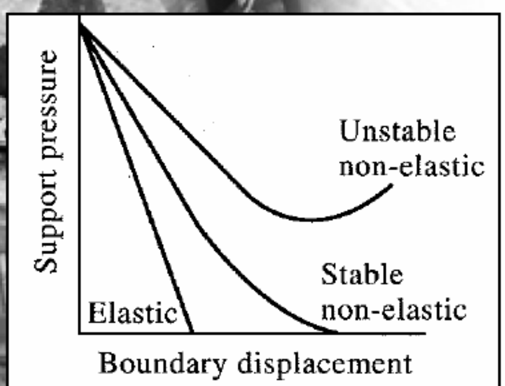


Ground Response Curve

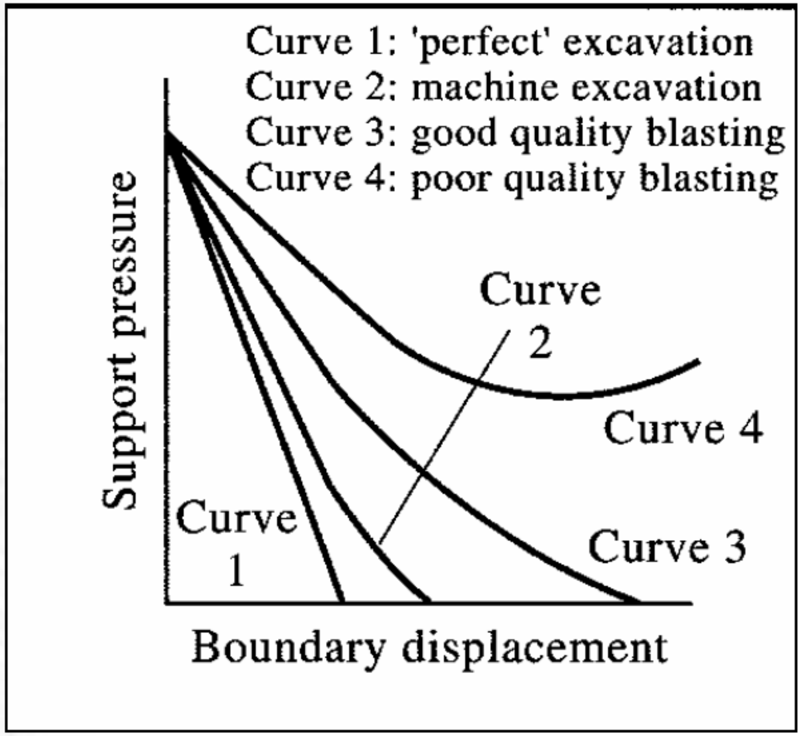
Considering the 'stable' non-elastic curve, the intersection of the curve with the boundary displacement axis occurs at a higher displacement value (e.g. up to 10% of the radius). Such a displacement is usually unacceptable in a rail tunnel, but may be tolerable in a temporary mine opening.





Finally, the 'unstable' non-elasticity curve indicates the definite need for support, because the curve does not intersect the boundary displacement axis, i.e. the opening will collapse without support.

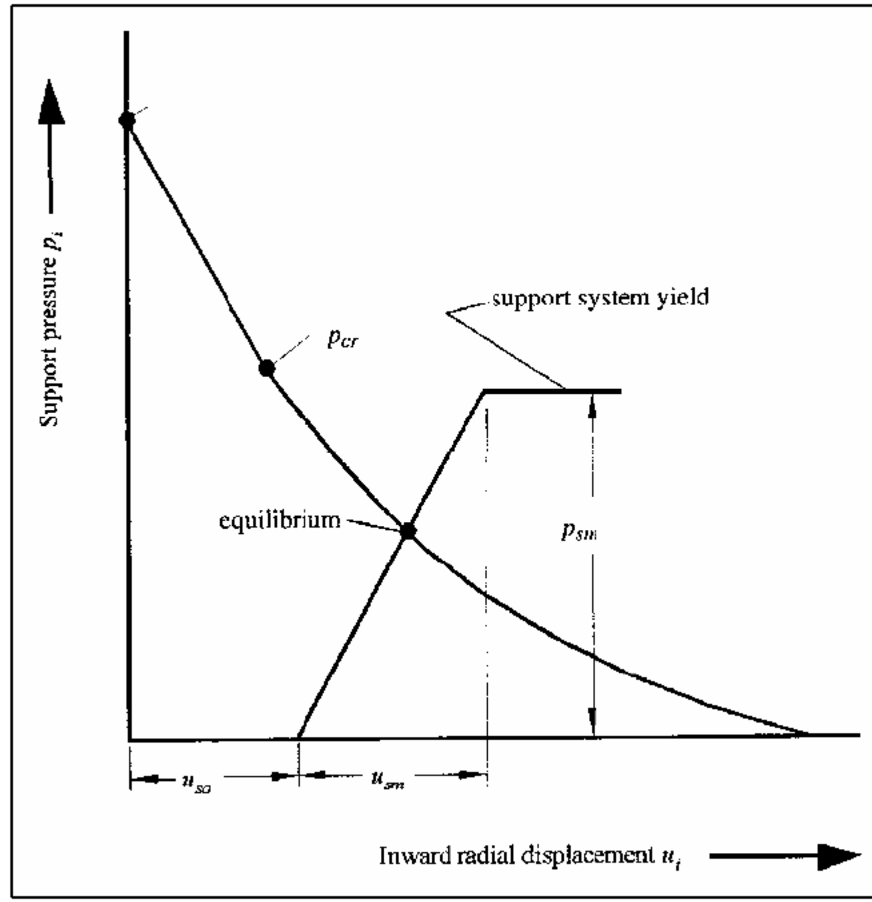
Ground Response Curve



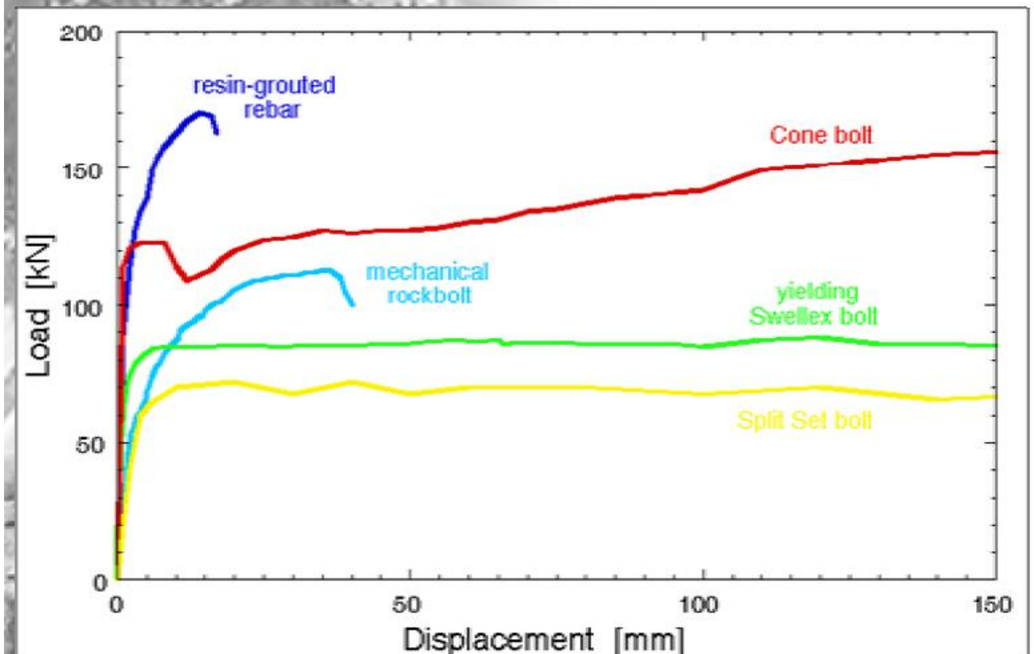
... effect of excavation methods on shape of the ground response curve due induced damage and alteration of rock mass properties.

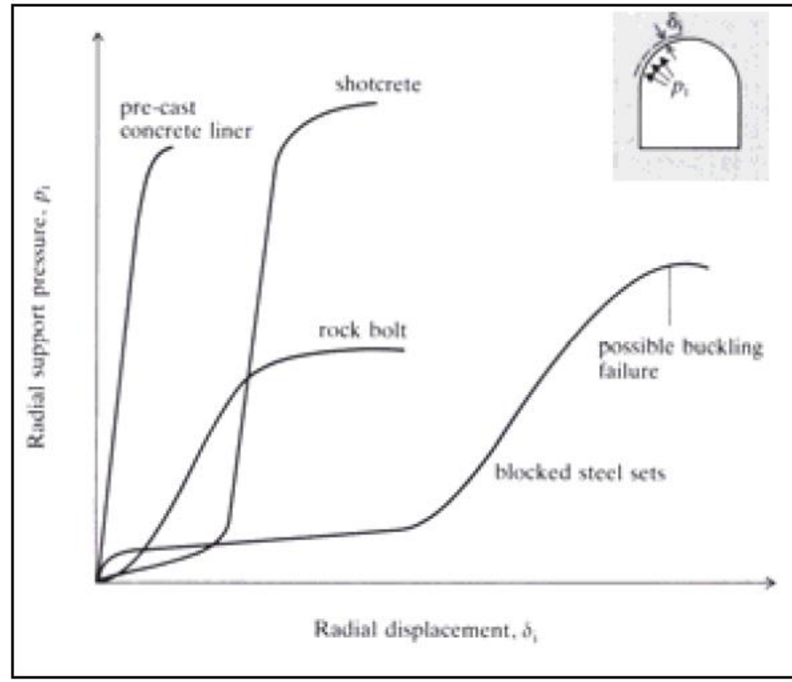
Support Reaction Curve

If support is required, we can gain an indication of the efficacy of particular support systems by plotting the elastic behaviour of the support, the available support line, on the same axes as the ground response curve. The points of interest are where the available support lines intersect the ground response curves: at these points, equilibrium has been achieved.



Support Reaction Curve

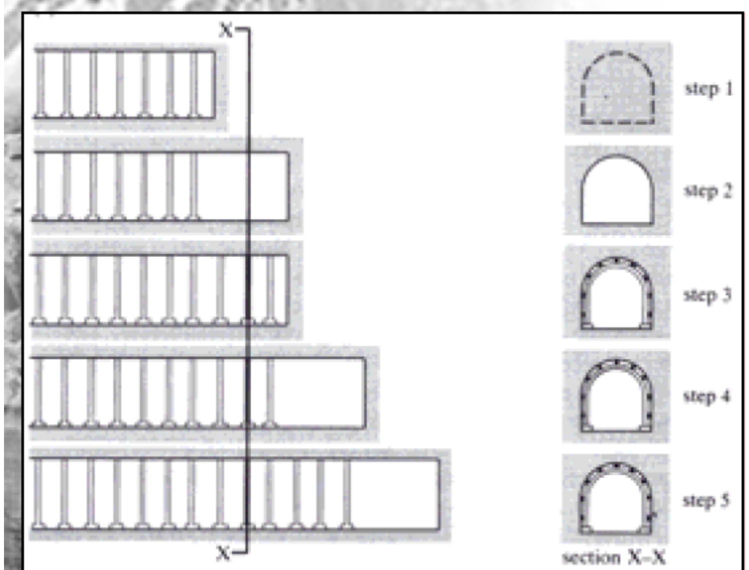


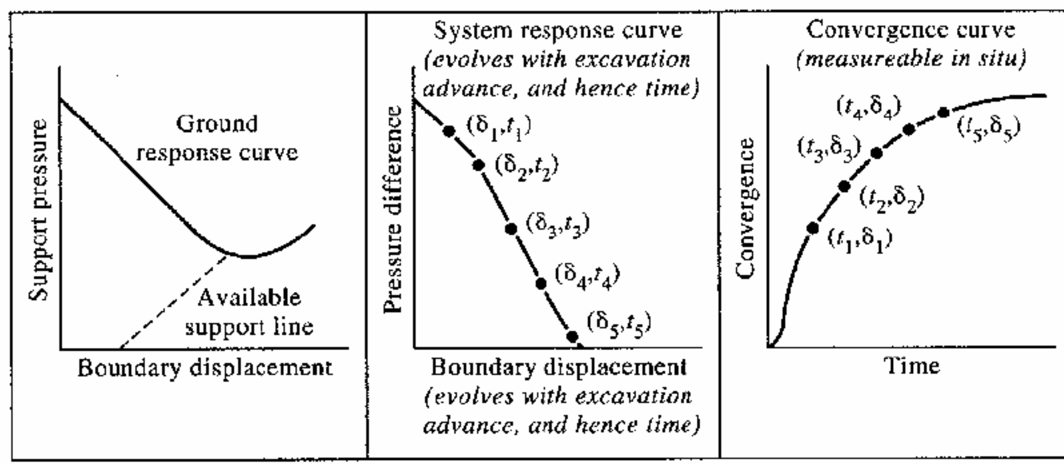


... examples of non-linear support reaction curves for various support types (after Kaiser et al. 1996; Brady & Brown 1993).

Rock-Support Interaction Analysis

Of practical significance, in relation to the ground support curve, is the fact that support cannot be installed contemporaneously with excavation, and so some initial displacement must occur before the support is installed. Thus, the available support line starts with a displacement offset.

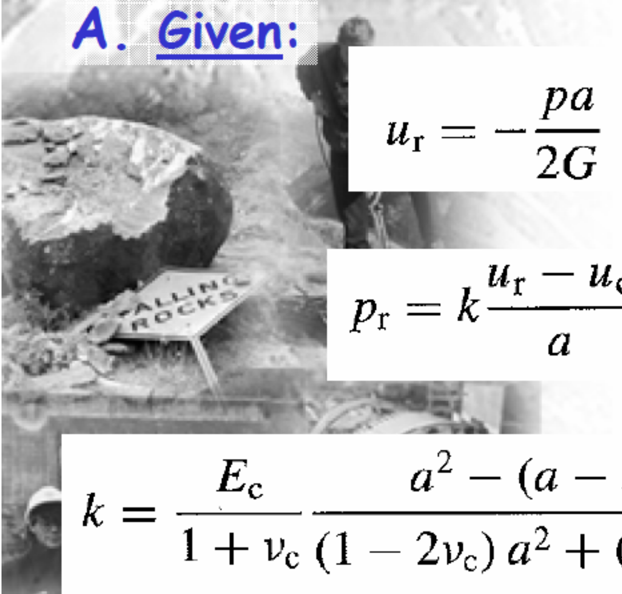




Example #3: Rock-Support Interaction Curves

Q. A circular tunnel of radius 1.85 m is excavated in rock subjected to an initial hydrostatic stress field of 20 MPa and provided with a concrete lining of internal radius 1.70 m. Assuming elastic behaviour of the rock/lining, calculate/plot the radial pressure and the radial displacement at the rock lining interface if the lining is installed after a radial displacement of 1 mm has occurred at the tunnel boundary.

A. Given:

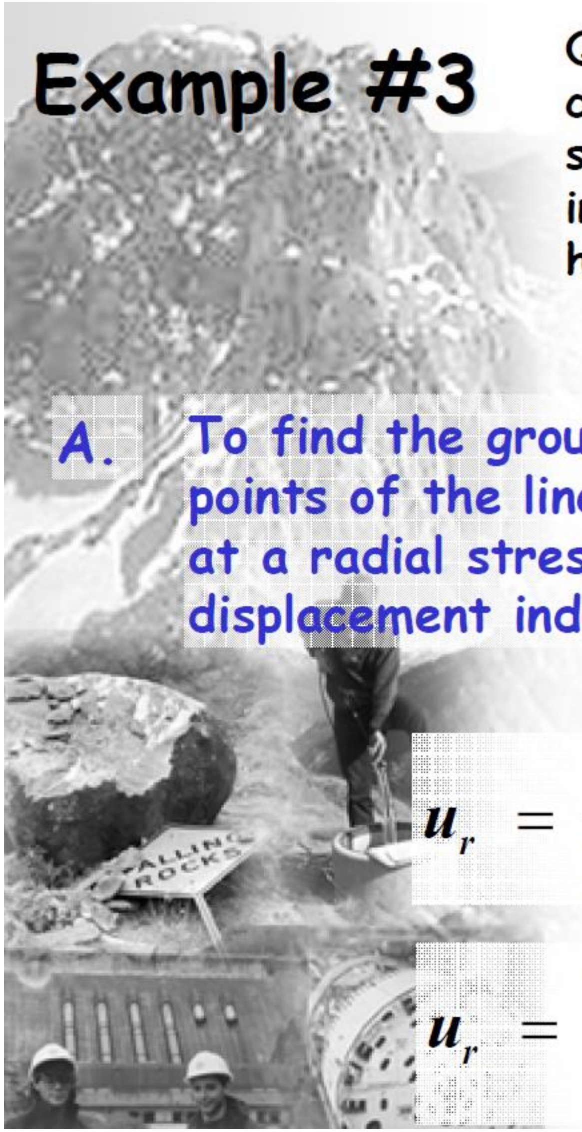


$$u_r = -\frac{pa}{2G}$$

$$p_r = k\frac{u_r - u_o}{a}$$

$$k = \frac{E_c}{1+\nu_c}\frac{a^2-(a-t_c)^2}{(1-2\nu_c)\,a^2+(a-t_c)^2}$$

p = hydrostatic stress
a = tunnel radius
G = shear modulus (assume 2 GPa)
p_r = radial support pressure
k = lining stiffness
u_o = rock displacement when support installed
t_c = concrete lining thickness
E_c = lining elastic modulus (assume 30 GPa)
ν_c = lining Poisson ratio (assume 0.25)



Example #3

Q. Calculate/plot the radial pressure and displacement curves for a 1.85 m radius tunnel subjected to a hydrostatic stress field of 20 MPa and provided with a concrete lining of internal radius 1.70 m installed after 1 mm of convergence has occurred.

A. To find the ground response curve we need to identify the two end points of the line: one is the *in situ* condition of zero displacement at a radial stress of 20 MPa, the other is the maximum elastic displacement induced when the radial stress is zero.

$$u_r = \frac{pa}{2G}$$

p = hydrostatic stress
a = tunnel radius
G = shear modulus (assume 2 GPa)

$$u_r = \frac{(20e6\,Pa)(1.85m)}{2\cdot(2e9\ Pa)} = 0.00925m$$

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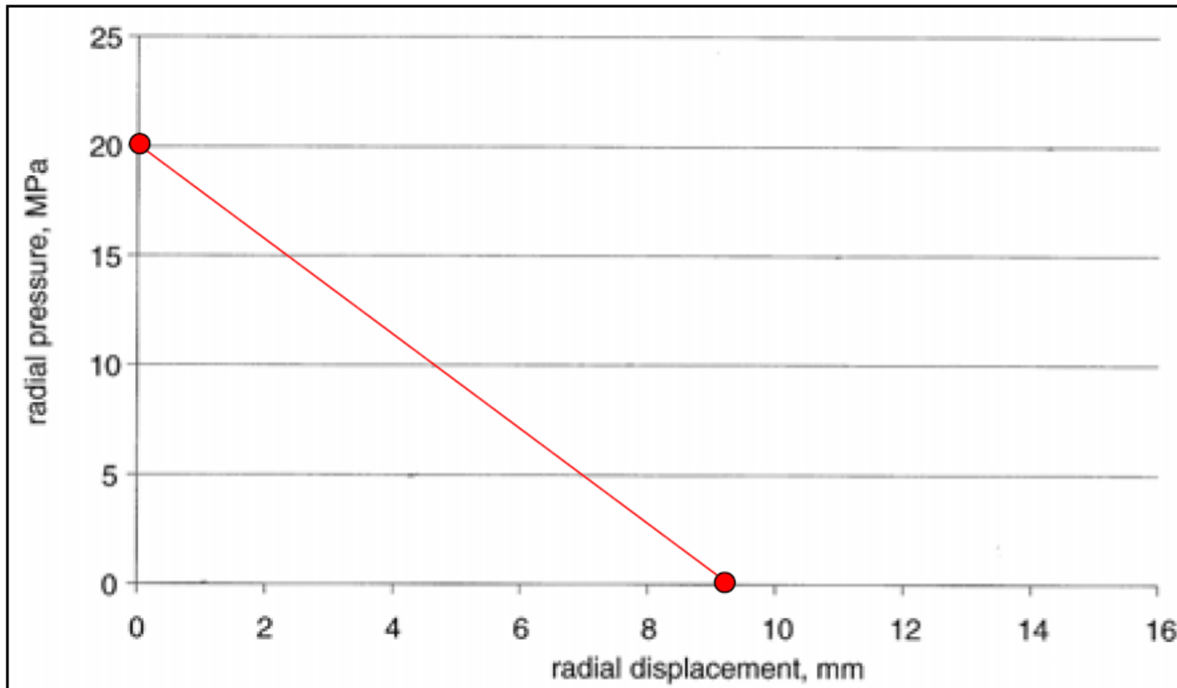
A. Plotting our ground response line, we have two known points:

1. $$p_r = 20\ MPa \quad u_r = 0\ mm$$

2. $$p_r = 0\ MPa \quad u_r = 9.25\ mm$$



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$$p_r = k \frac{u_r - u_o}{a}$$

$$k = \frac{E_c}{1+\nu_c} \frac{a^2 - (a - t_c)^2}{(1 - 2\nu_c)\, a^2 + (a - t_c)^2}$$

p_r = radial support pressure
k = lining stiffness
u_o = rock displacement when support installed
t_c = concrete lining thickness
E_c = lining elastic modulus (assume 30 GPa)
ν_c = lining Poisson ratio (assume 0.25)

Example #3

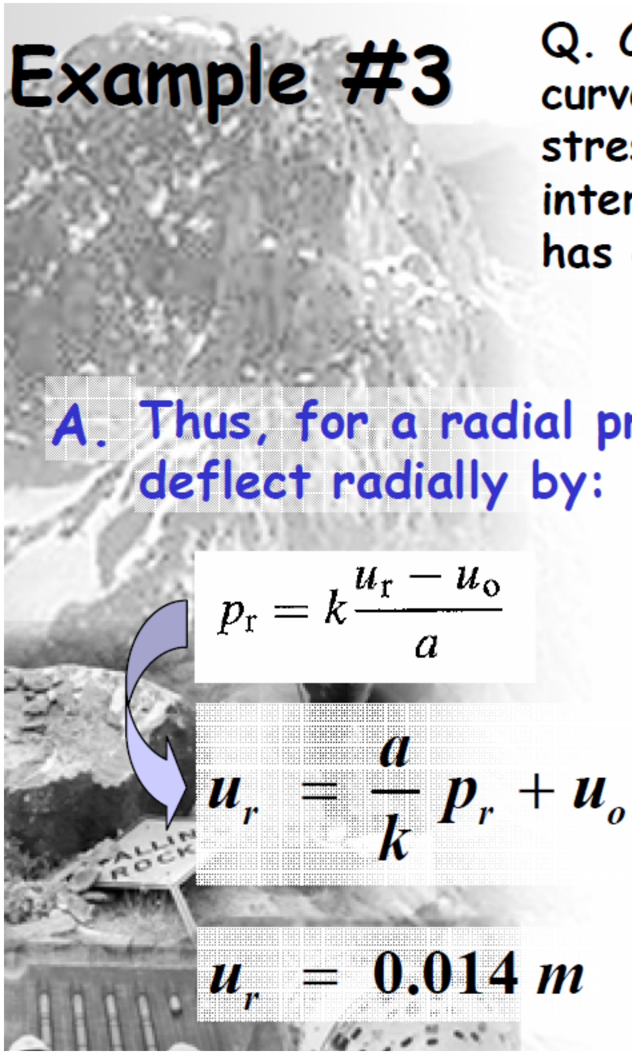
Q. Calculate/plot the radial pressure and displacement curves for a 1.85 m radius tunnel subjected to a hydrostatic stress field of 20 MPa and provided with a concrete lining of internal radius 1.70 m installed after 1 mm of convergence has occurred.

A. Solving for the stiffness of the lining, where t_c = 1.85 - 1.70 = 0.15 m, E_c = 30 GPa and ν_c = 0.25, we get:

$$k = \frac{E_c}{1+\nu_c} \frac{a^2 - (a - t_c)^2}{(1 - 2\nu_c)\, a^2 + (a - t_c)^2}$$

$$k = \frac{30\,GPa}{1+0.25}\left[\frac{(1.85m)^2 - (1.85m - 0.15m)^2}{(1-0.5)(1.85m)^2 + (1.85m - 0.15m)^2}\right]$$

$$k = \underline{\underline{2.78\,GPa}}$$



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A. Thus, for a radial pressure of 20 MPa and u_o = 1 mm, the lining will deflect radially by:

$$p_r = k \frac{u_r - u_o}{a}$$

$$u_r = \frac{a}{k} p_r + u_o = \frac{1.85m}{2.78e9\ Pa} 20e6\ Pa + 0.001\ m$$

$$u_r = 0.014\ m$$

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Q. Calculate/plot the radial pressure and displacement curves for a 1.85 m radius tunnel subjected to a hydrostatic stress field of 20 MPa and provided with a concrete lining of internal radius 1.70 m installed after 1 mm of convergence has occurred.

A. Plotting our support reaction line, we have two known points:

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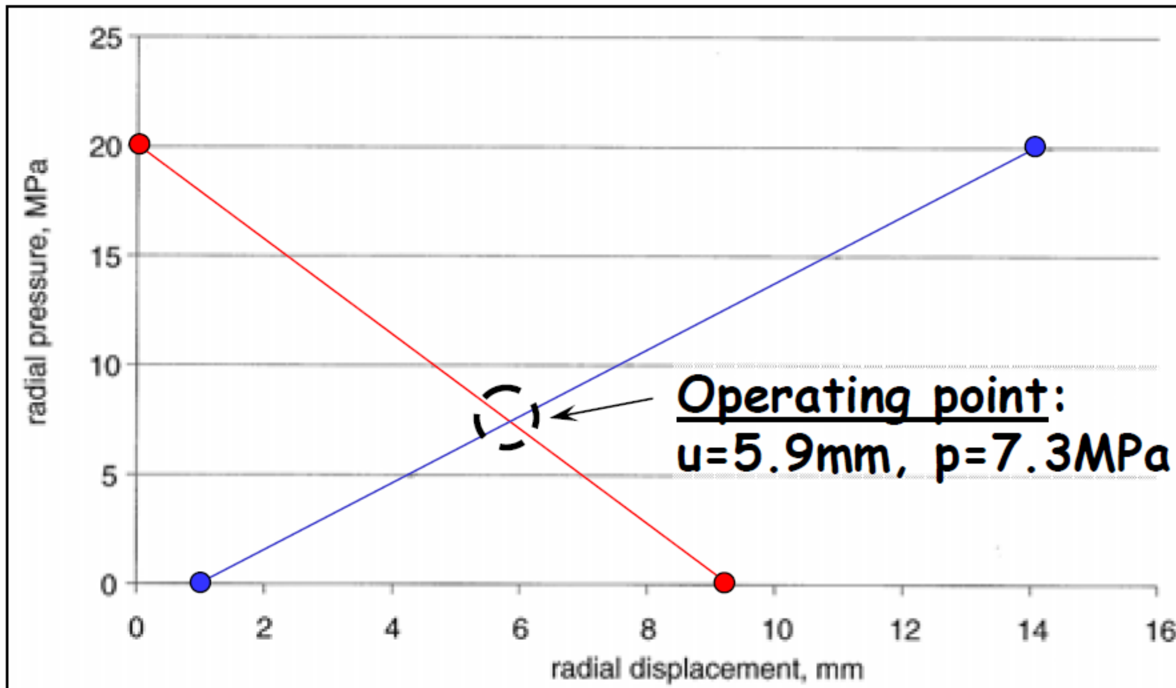

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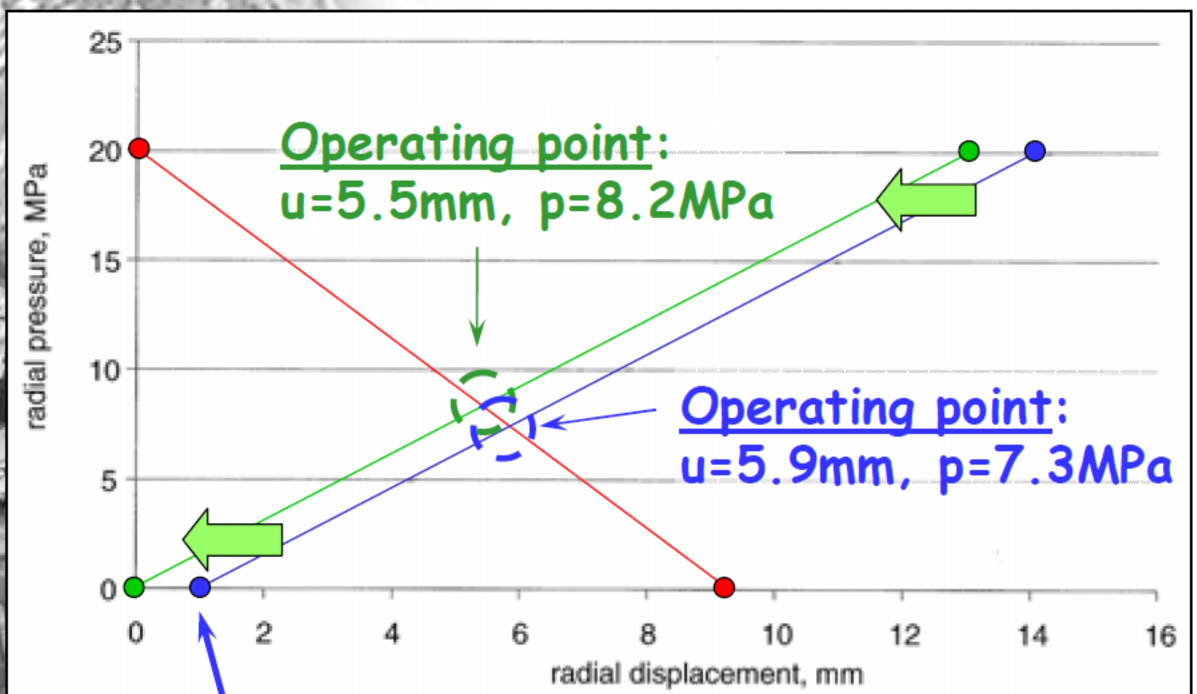
A.

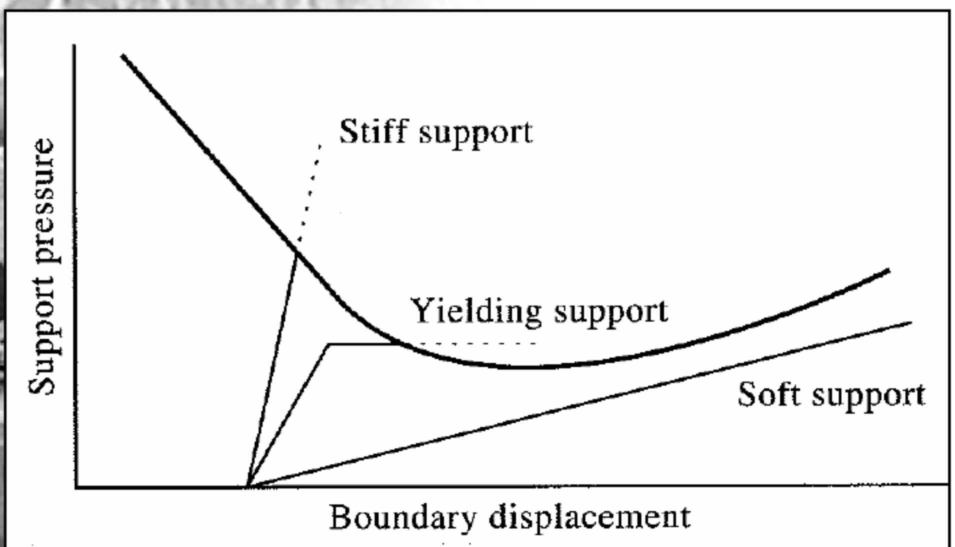


1 mm displacement of tunnel boundary

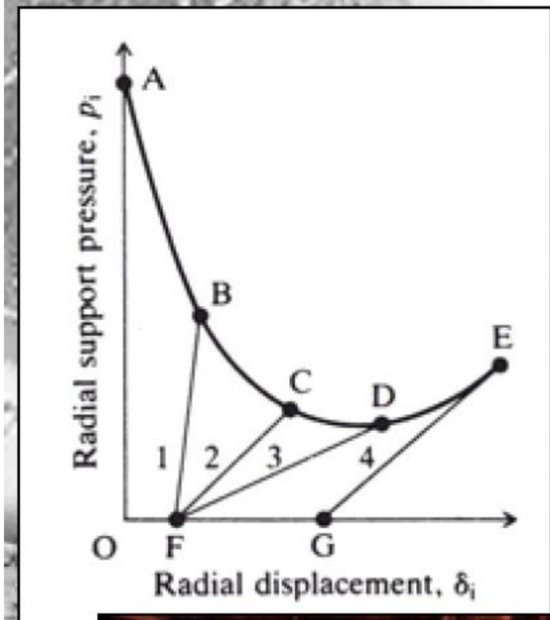
This shows how, by delaying the installation of the lining, we have reduced the pressure it is required to withstand – but at the expense of increasing the final radial displacement.

Rock Support in Yielding Rock

Another important conclusion drawn from these curves, for the case of unstable non-elastic conditions, is that stiff support (e.g. pre-cast concrete segments) may be successful, but that soft support (e.g. steel arches) may not bring the system to equilibrium.



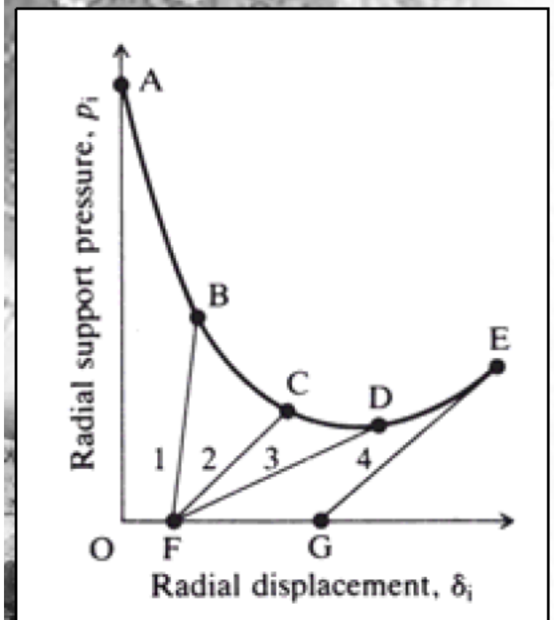
Rock Support in Yielding Rock



Support 1 is installed at *F* and reaches equilibrium with the rock mass at point *B*:

This support is too stiff for the purpose and attracts an excessive share of the redistributed load. As a consequence, the support elements may fail causing catastrophic failure of the rock surrounding the excavation.

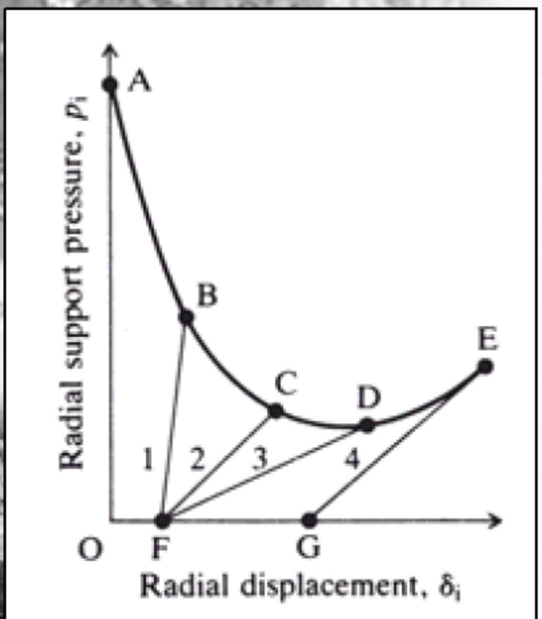
Rock Support in Yielding Rock



Support 2, having a lower stiffness, is installed at *F* and reaches equilibrium with the rock mass at point *C*:

Provided the corresponding convergence of the excavation is acceptable operationally, this system provides a good solution. The rock mass carries a major portion of the redistributed load, and the support elements are not stressed excessively. Note that if this support was temporary and was to be removed after equilibrium had been reached, uncontrolled displacement and collapse of the rock mass would almost certainly occur.

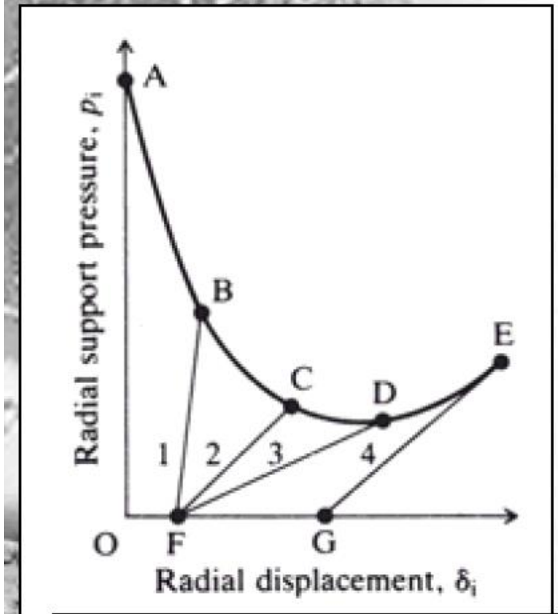
Rock Support in Yielding Rock



Support 3, having a much lower stiffness than support 2, is also installed at *F* but reaches equilibrium with the rock mass at point *D* where the rock mass has started to loosen:

Although this may provide an acceptable temporary solution, the situation is a dangerous one because any extra load imposed, for example by a redistribution of stress associated with the excavation of a nearby opening, will have to be carried by the support elements. In general, support 3 is too compliant for this particular application.

Rock Support in Yielding Rock



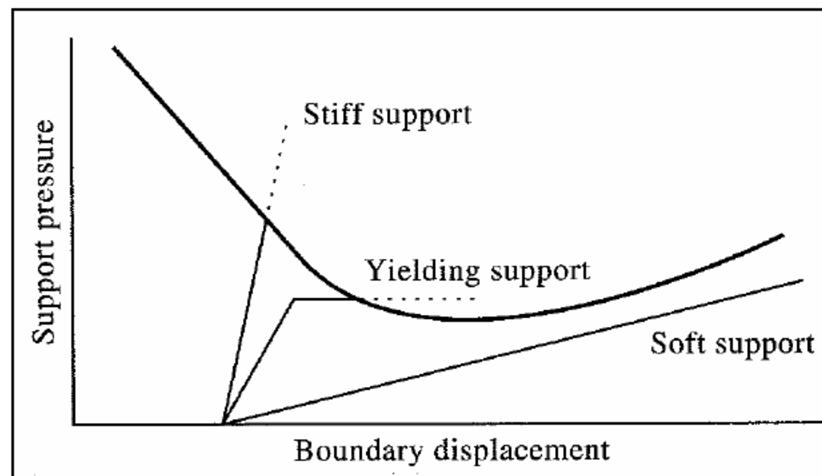
Support 4, of the same stiffness as support 2, is not installed until a radial displacement of the rock mass of *OG* has occurred. :

In this case, the support is installed late, excessive convergence of the excavation will occur, and the support elements will probably become overstressed before equilibrium is reached.

Rock Support in Yielding Rock

Thus, it should never be attempted to achieve zero displacement by introducing as stiff a support system as possible – this is never possible, and will also induce unnecessarily high support pressures. The support should be in harmony with the ground conditions, with the result that an optimal equilibrium position is achieved.

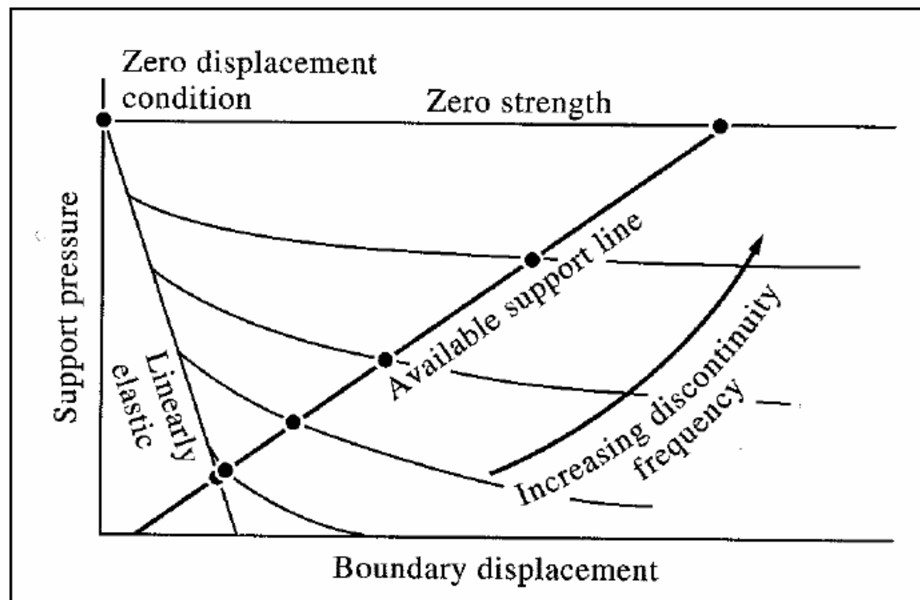
Thus, it is better to allow the rock to displace to some extent and then ensure equilibrium is achieved before any deleterious displacement of the rock occurs.



Rock Support in Highly Jointed Rock

A directly analogous ground response curve approach can be considered for the use of rock support in discontinuous rock. As the rock becomes more and more fractured with the attendant loss of strength, the ground response curve becomes progressively flatter. This effect is similar to the reduction in rock mass modulus with increasing discontinuity frequency.

The two limiting cases of the suite of ground response curves are linear elastic behaviour and zero strength. In between, it can be seen that increasingly higher support pressures are required for equilibrium with increasing discontinuity frequency.

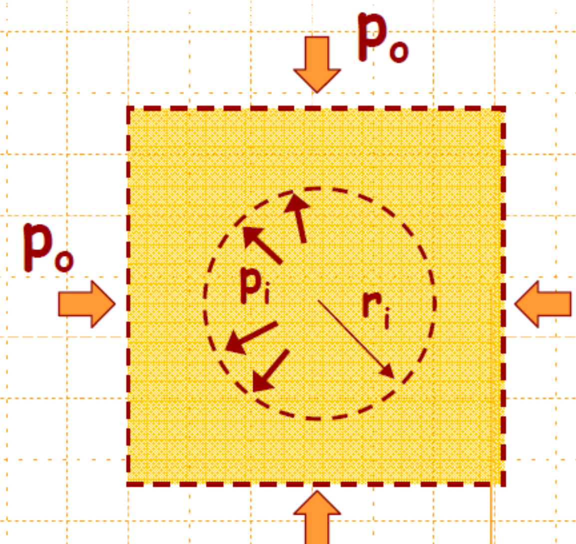


Rock Support Principles

Consider a tunnel being advanced by conventional methods, where steel sets are installed after each drill & blast cycle.

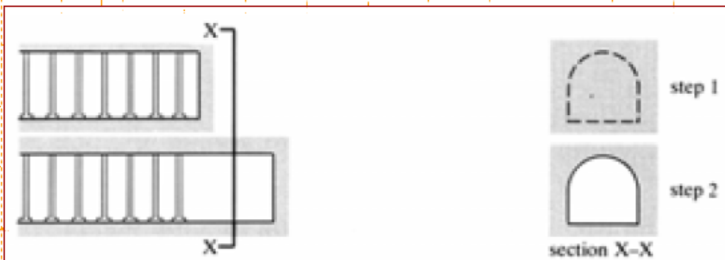
Brady & Brown (1993)

In Step 1: the heading has not reached X-X and the rock mass on the periphery of the future tunnel profile is in equilibrium with the internal pressure (p_i) acting equal and opposite to p_o.

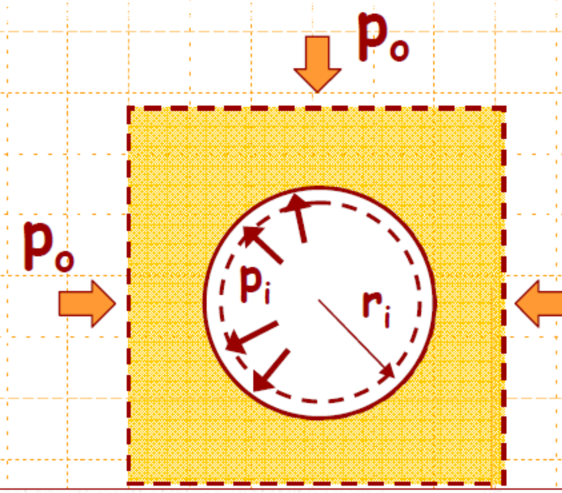


Rock Support Principles

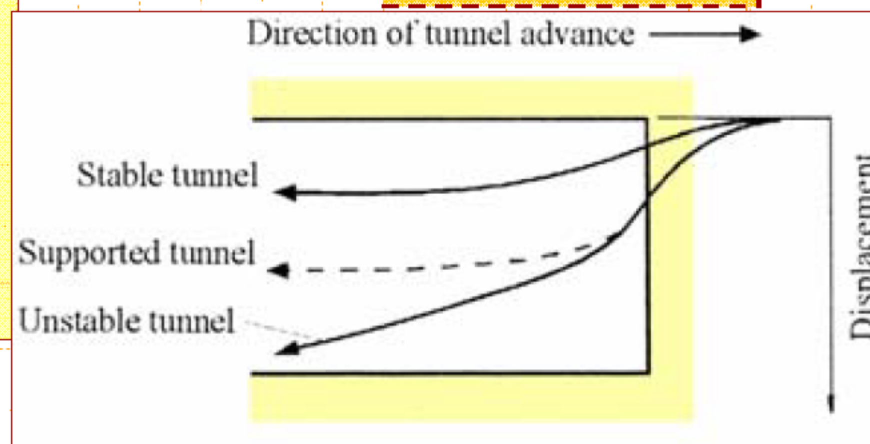
Consider a tunnel being advanced by conventional methods, where steel sets are installed after each drill & blast cycle.



Brady & Brown (1993)



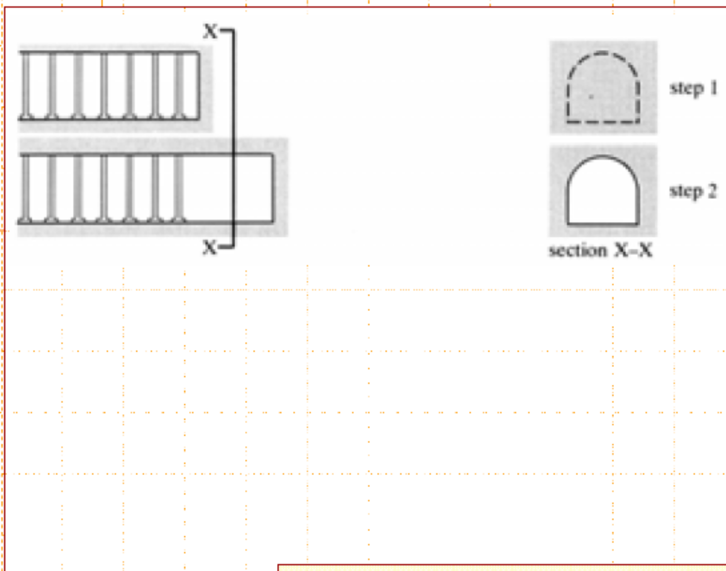
In Step 2: the face has advanced beyond X-X and the support pressure (p_i) provided by the rock inside the tunnel has been reduced to zero. As the blasted rock must be removed before the steel sets (support) can be installed, deformation of the excavation boundaries starts to occur.

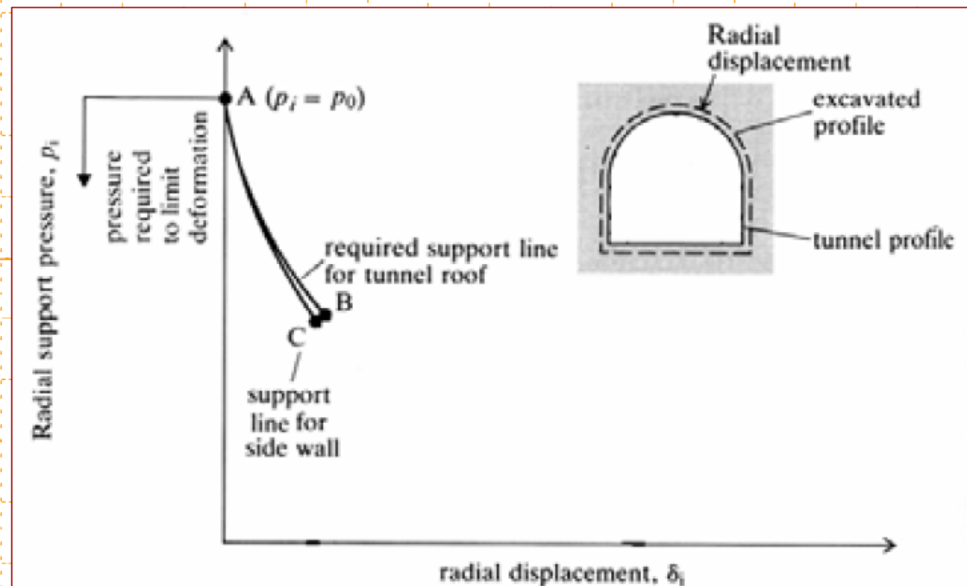


Tunnel Support Principles

We can then plot the radial support pressure (p_i) required to limit the boundary displacement (δ_i) to a given value.

Brady & Brown (1993)





Thus, by advancing the excavation and removing the internal support pressure provided by the face, the tunnel roof will converge and displace along line AB (or AC in the case of the tunnel walls; the roof deformation follows a different path due to the extra load imposed by gravity on the loosened rock in the roof).

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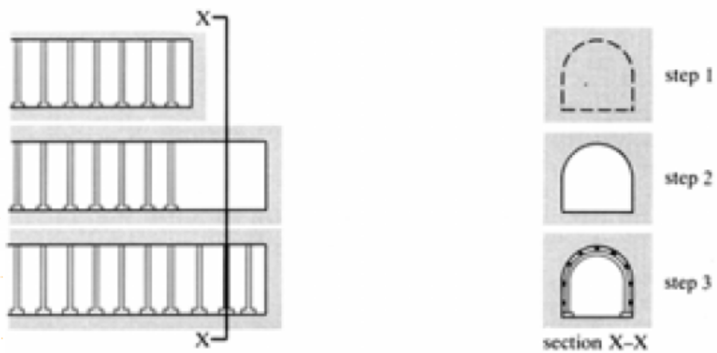

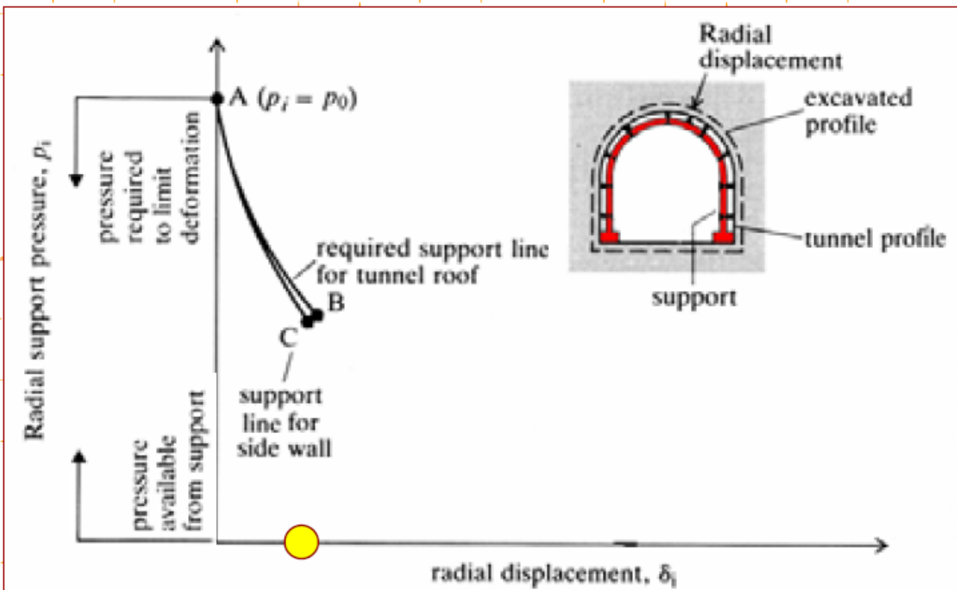

By **Step 3**: the heading has been mucked out and steel sets have been installed close to the face. From this point on, any deformation of the tunnel roof or walls will result in loading of the steel sets.

Tunnel Support Principles

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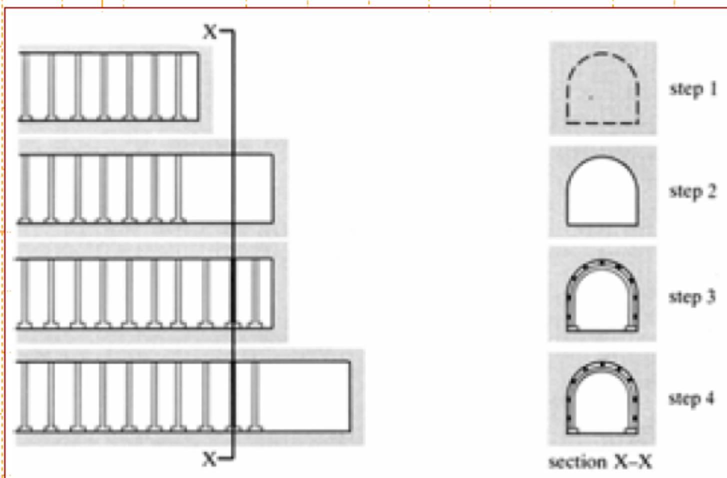
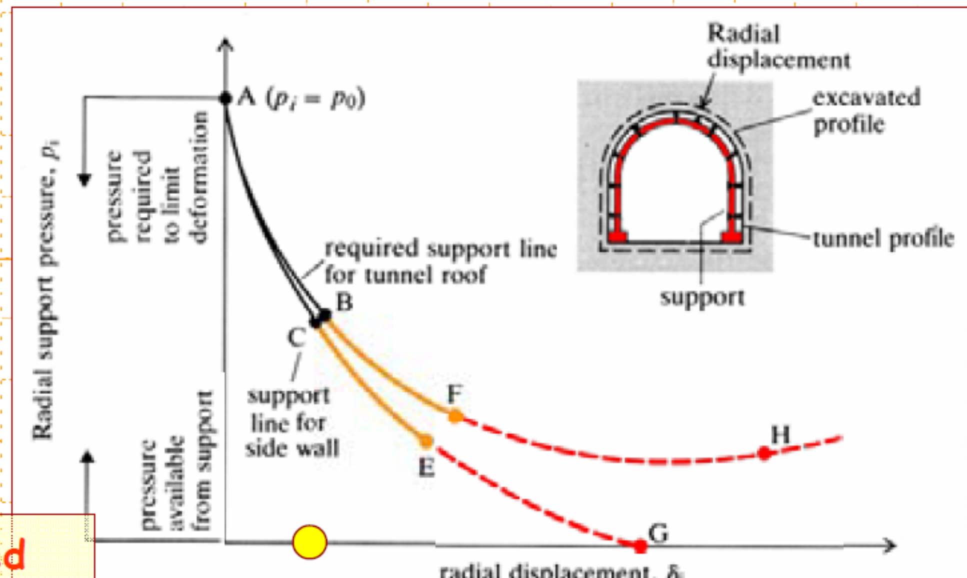
Brady & Brown (1993)





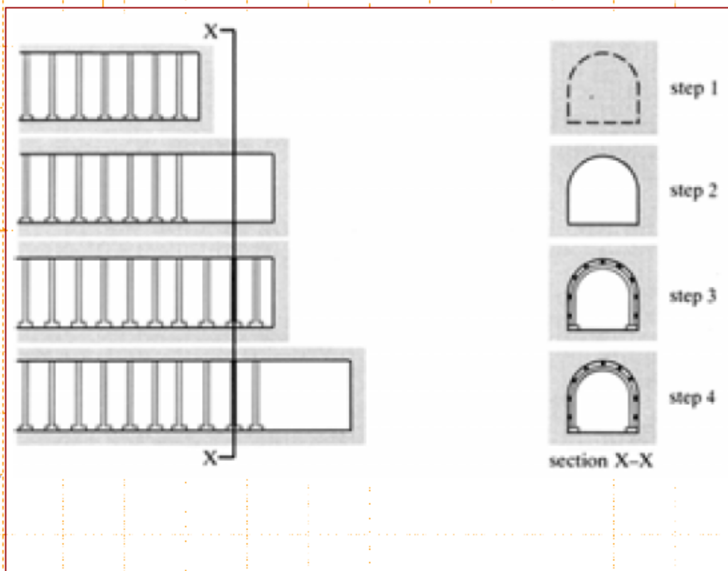
In Step 4: the heading is advanced one and a half tunnel diameters beyond X-X by another blast. The restraint offered by the proximity of the face is now negligible and further convergence of the tunnel boundaries occurs.

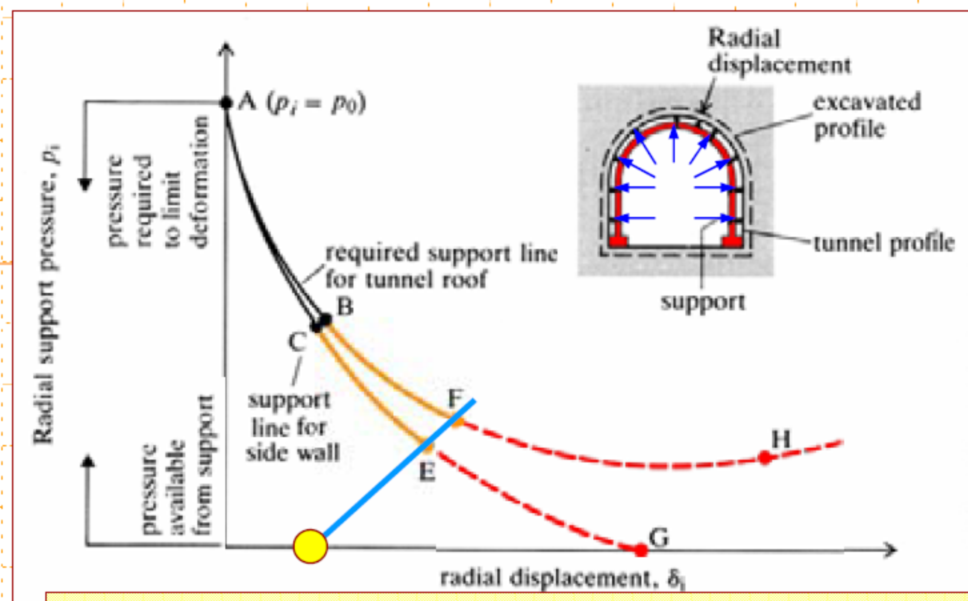
If steel sets had not been installed, the radial displacements at X-X would continue increasing along the dashed lines EG and FH. In this case, the side walls would reach equilibrium at point G. However, the roof would continue deforming until it failed.

Tunnel Support Principles

We can then plot the radial support pressure (p_i) required to limit the boundary displacement (δ_i) to a given value.

Brady & Brown (1993)





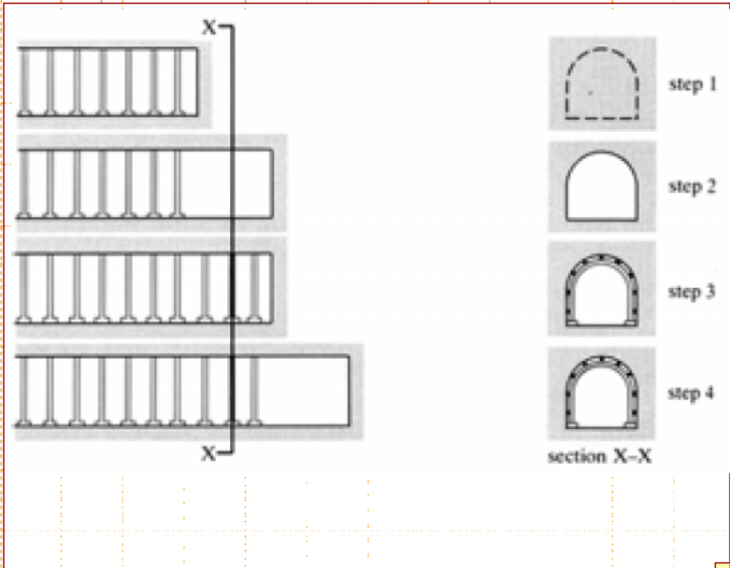
… but with steel sets installed, the tunnel convergence will begin to load the support.

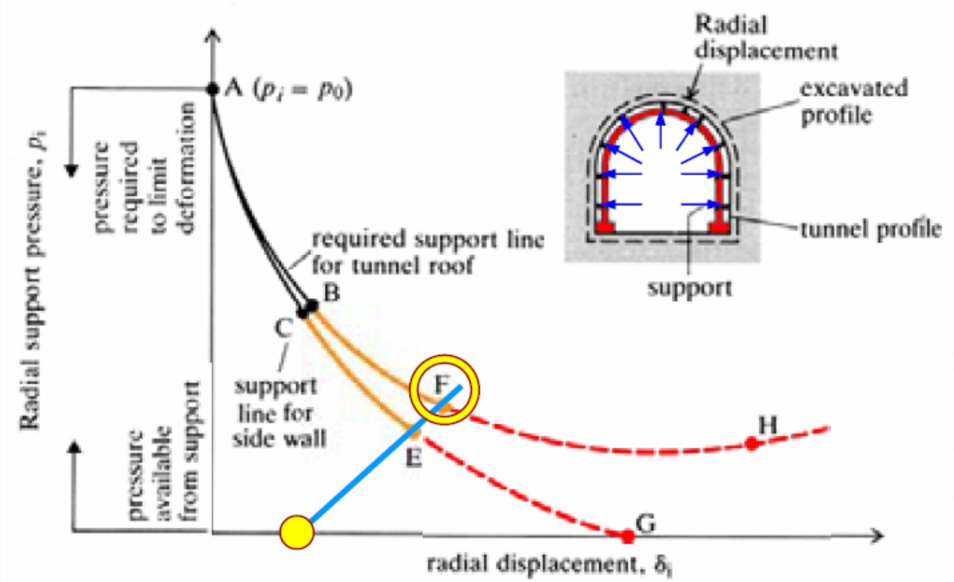
This load path is known as the <u>support reaction line</u> (or available support line). The curve representing the behaviour of the rock mass is known as the <u>ground response curve</u> (or support required curve).

Tunnel Support Principles

We can then plot the radial support pressure (p_i) required to limit the boundary displacement (δ_i) to a given value.

Brady & Brown (1993)





Equilibrium between the rock and steel sets is reached where the lines intersect.

It is important to note that most of the redistributed stress arising from the excavation is carried by the rock and not by the steel sets!!

Example 3: Rock-Support Interaction Curves

Q. A circular tunnel of radius 1.85 m is excavated in rock subjected to an initial hydrostatic stress field of 20 MPa and provided with a concrete lining of internal radius 1.70 m. Assuming elastic behaviour of the rock/lining, calculate/plot the radial pressure and the radial displacement at the rock lining interface if the lining is installed after a radial displacement of 1 mm has occurred at the tunnel boundary.

A. Given:

$$u_r = -\frac{pa}{2G}$$

$$p_r = k\frac{u_r - u_o}{a}$$

$$k = \frac{E_c}{1+\nu_c}\frac{a^2-(a-t_c)^2}{(1-2\nu_c)\,a^2+(a-t_c)^2}$$

p = hydrostatic stress
a = tunnel radius
G = shear modulus (assume 2 GPa)
p_r = radial support pressure
k = lining stiffness
u_o = rock displacement when support installed
t_c = concrete lining thickness
E_c = lining elastic modulus (assume 30 GPa)
ν_c = lining Poisson ratio (assume 0.25)

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$$u_r = \frac{pa}{2G}$$

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$$u_r = \frac{(20e6Pa)(1.85m)}{2 \cdot (2e9\ Pa)} = 0.00925m$$

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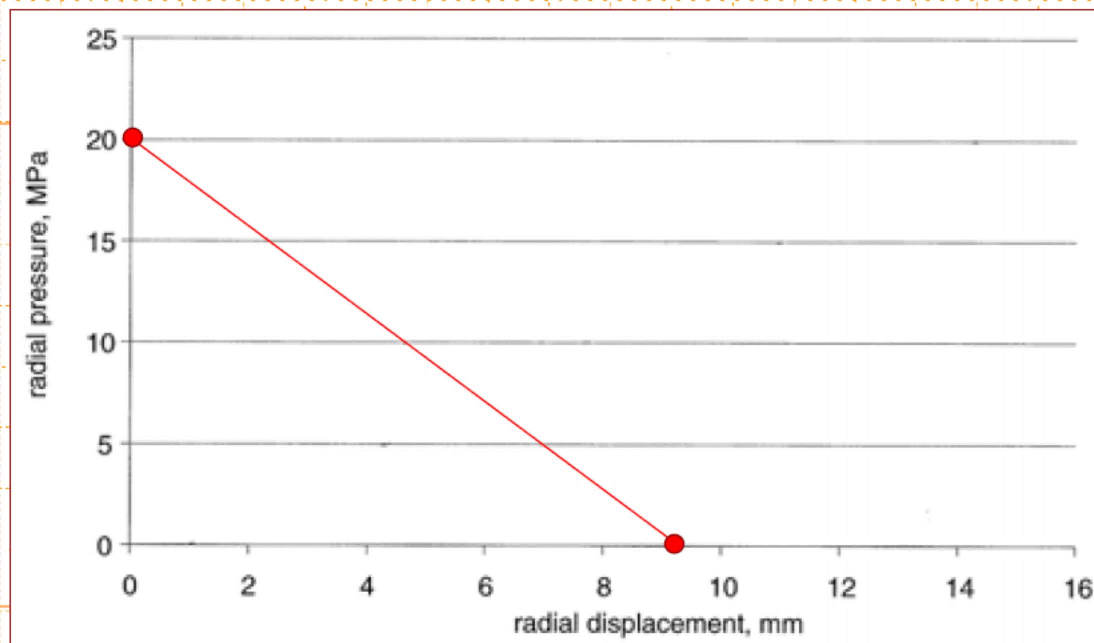

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$$k = \frac{E_c}{1+\nu_c}\,\frac{a^2-(a-t_c)^2}{(1-2\nu_c)\,a^2+(a-t_c)^2}$$

$$k = \frac{30\,GPa}{1+0.25}\left[\frac{(1.85m)^2-(1.85m-0.15m)^2}{(1-0.5)(1.85m)^2+(1.85m-0.15m)^2}\right]$$

$$k = \underline{\underline{2.78\,GPa}}$$

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   $u_r = 1\ mm$

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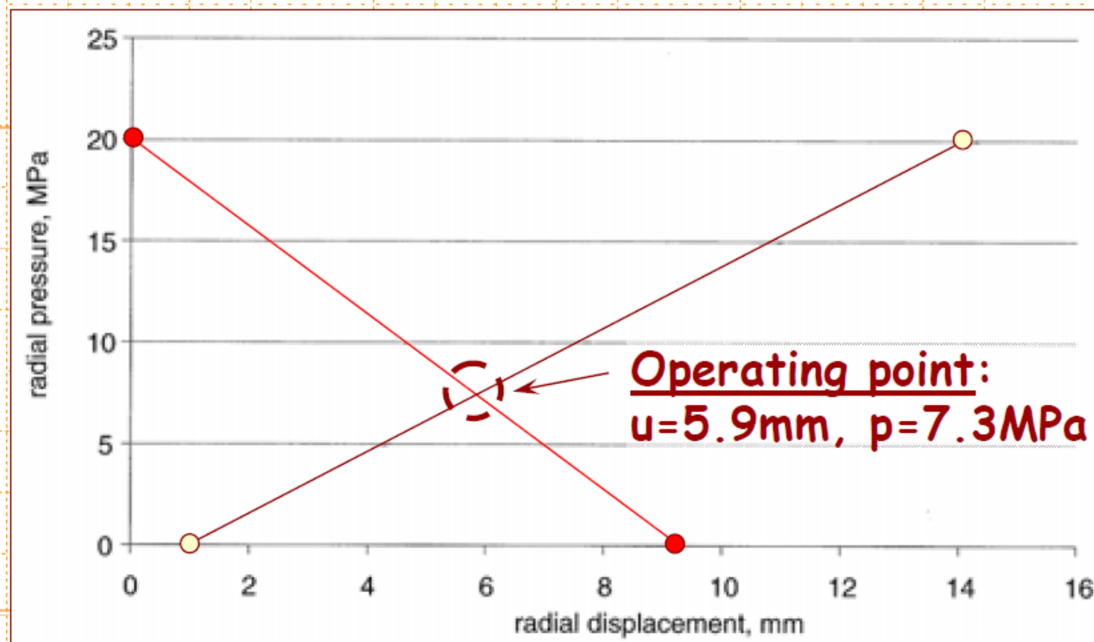

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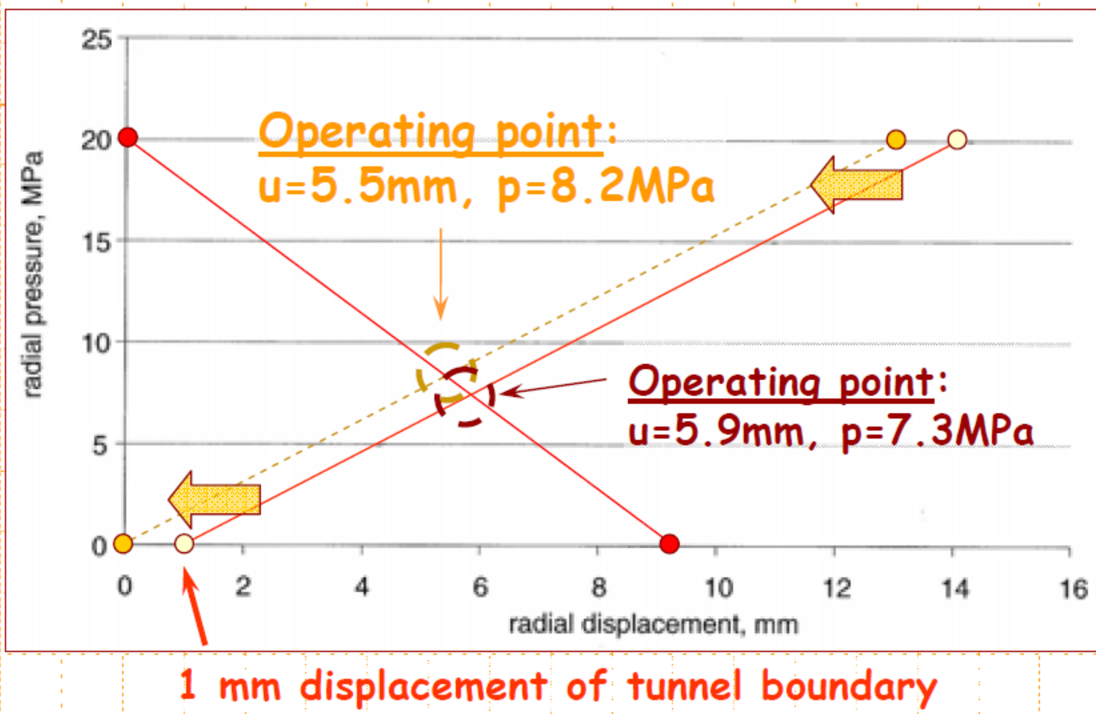
A.

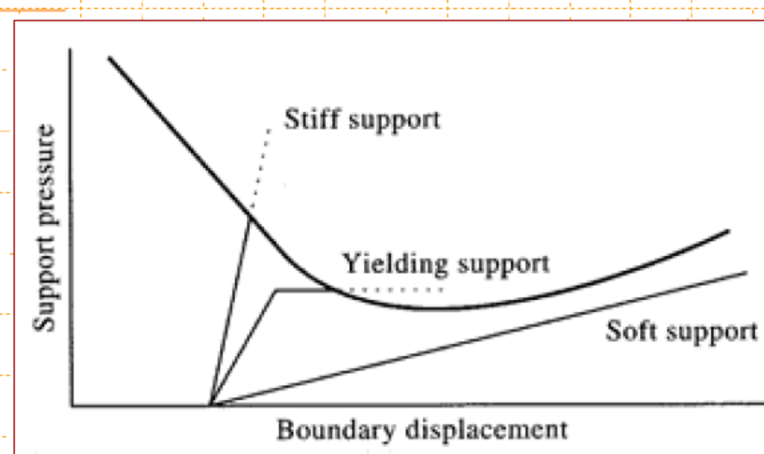


This shows how, by delaying the installation of the lining, we have reduced the pressure it is required to withstand – but at the expense of increasing the final radial displacement.

Rock Support in Yielding Rock

Thus, it should never be attempted to achieve zero displacement by introducing as stiff a support system as possible - this is never possible, and will also induce unnecessarily high support pressures. The support should be in harmony with the ground conditions, with the result that an optimal equilibrium position is achieved.

In general, it is better to allow the rock to displace to some extent and then ensure equilibrium is achieved before any deleterious displacement of the rock occurs.



Hudson & Harrison (1997)

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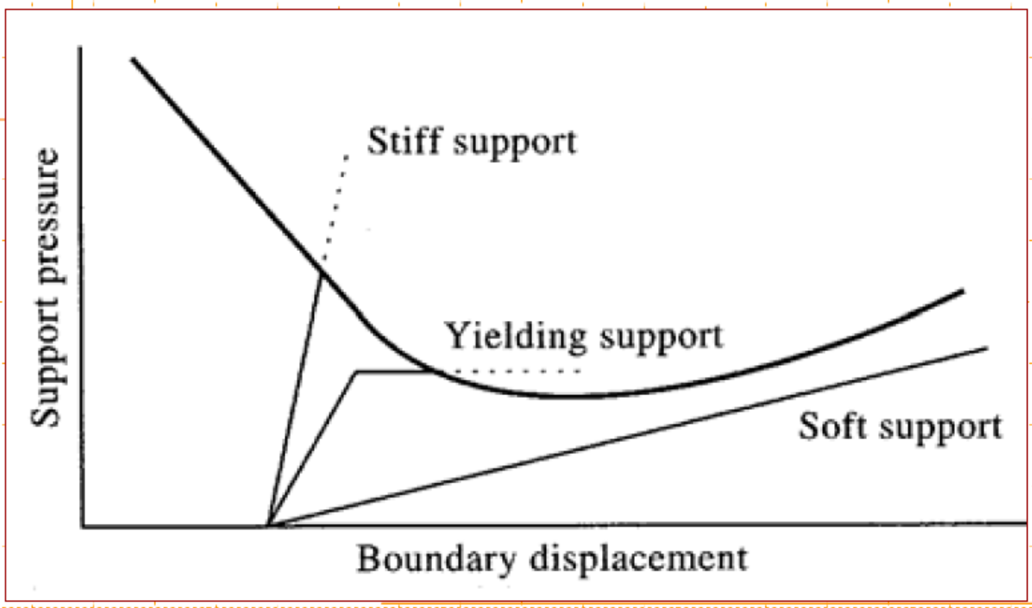
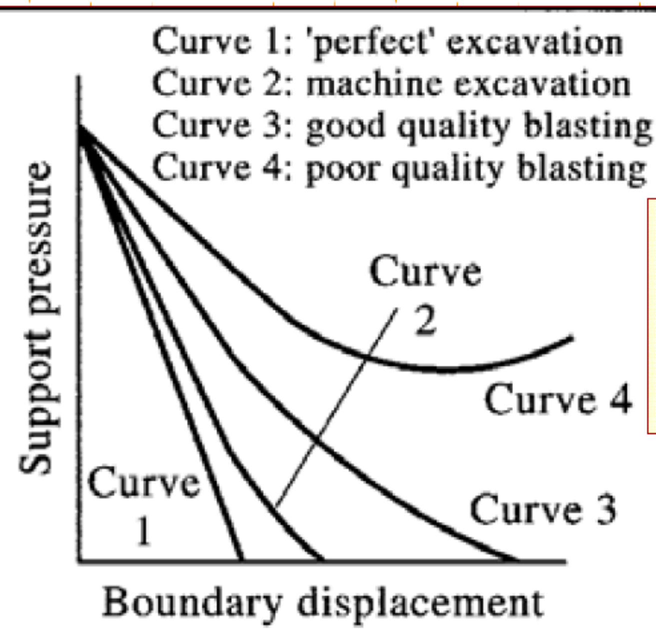


Ground Response Curve – Yielding Rock

It should also be noted that plastic failure of the rock mass does not necessarily mean that the tunnel will collapse. The yielded material may still have considerable strength and, provided that the plastic zone is small compared with the tunnel radius, the only evidence of failure may be a few fresh cracks and some minor spalling. In contrast, when a large plastic zone forms, large inward displacements may occur which may lead to the loosening and collapse of the tunnel.

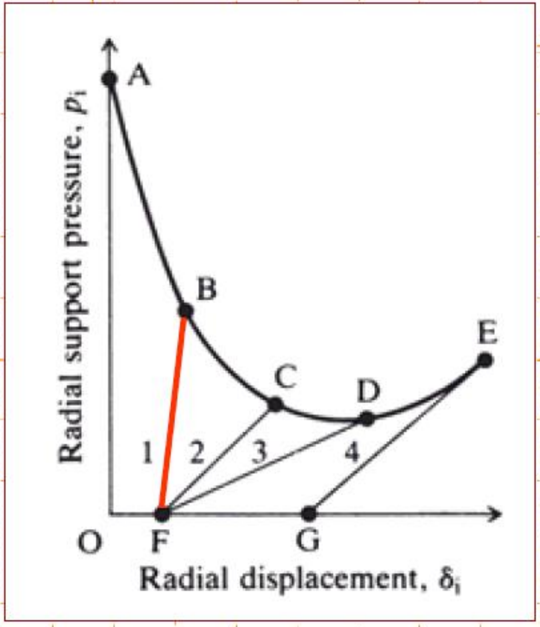


Hudson & Harrison (1997)

Effect of excavation methods on shape of the ground response curve due induced damage and alteration of rock mass properties.

The primary function of support is to control the inward displacement of the walls to prevent loosening.

Rock Support in Yielding Rock



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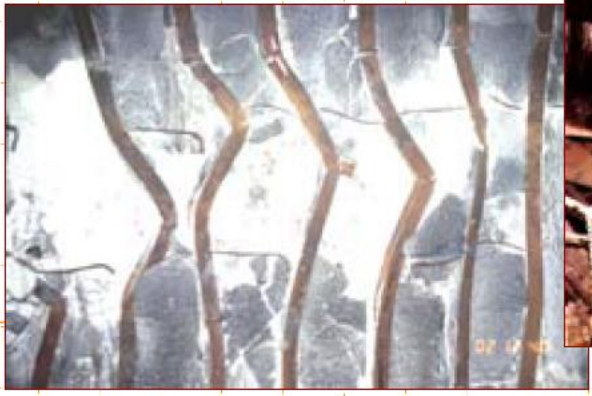
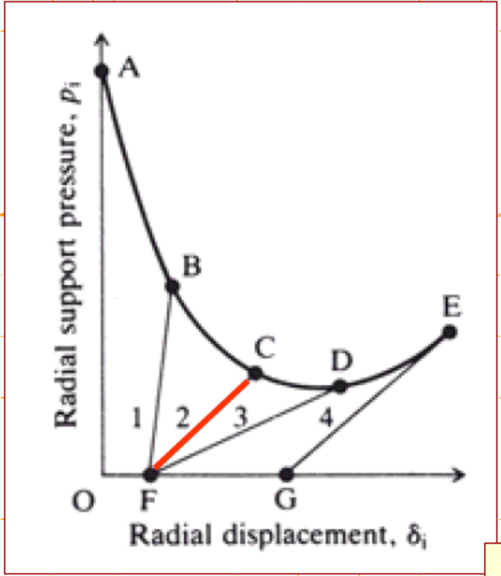
This support is too stiff for the purpose and attracts an excessive share of the redistributed load. As a consequence, the support elements may fail causing catastrophic failure of the rock surrounding the excavation.



Rock Support in Yielding Rock

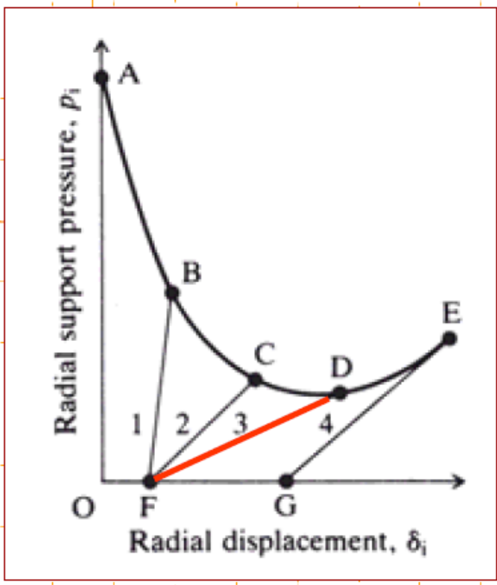


Support 2, having a lower stiffness, is installed at *F* and reaches equilibrium with the rock mass at point *C*:

Provided the corresponding convergence of the excavation is acceptable operationally, this system provides a good solution. The rock mass carries a major portion of the redistributed load, and the support elements are not stressed excessively.

Note that if this support was temporary and was to be removed after equilibrium had been reached, uncontrolled displacement and collapse of the rock mass would almost certainly occur.

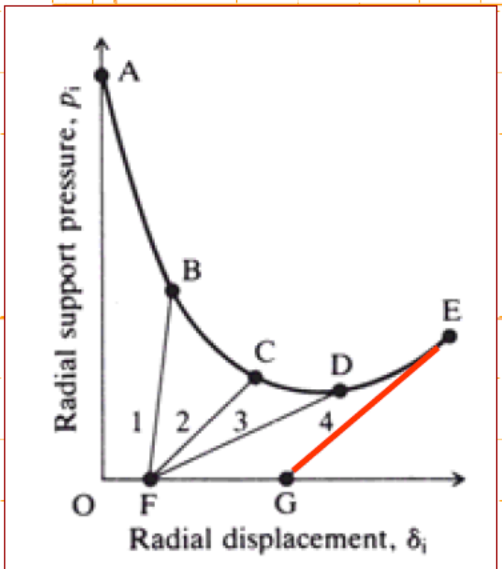
Rock Support in Yielding Rock



Support 3, having a much lower stiffness than support 2, is also installed at *F* but reaches equilibrium with the rock mass at point *D* where the rock mass has started to loosen:

Although this may provide an acceptable temporary solution, the situation is a dangerous one because any extra load imposed, for example by a redistribution of stress associated with the excavation of a nearby opening, will have to be carried by the support elements. In general, support 3 is too compliant for this particular application.

Rock Support in Yielding Rock



Support 4, of the same stiffness as support 2, is not installed until a radial displacement of the rock mass of *OG* has occurred. :

In this case, the support is installed late, excessive convergence of the excavation will occur, and the support elements will probably become overstressed before equilibrium is reached.

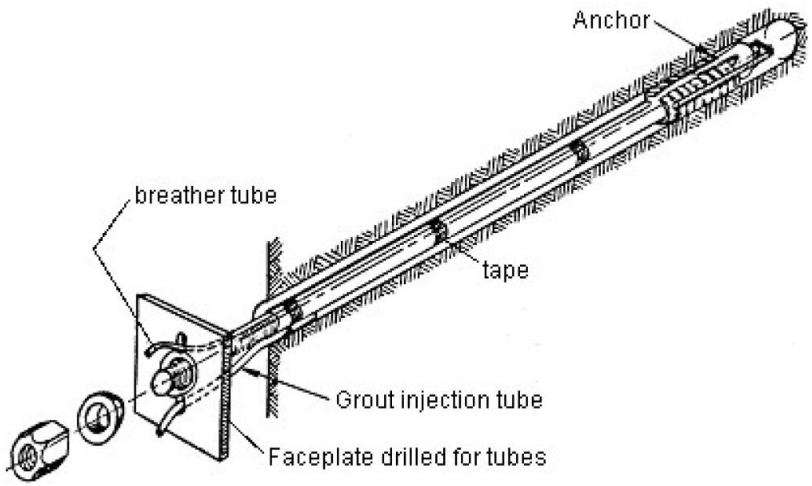


Figure 2: Mechanically anchored rockbolts of the type used on the Rio Grande project. These bolts were tensioned to 70% of their yield load upon installation and then, at a later stage, were re-tensioned and fully grouted.

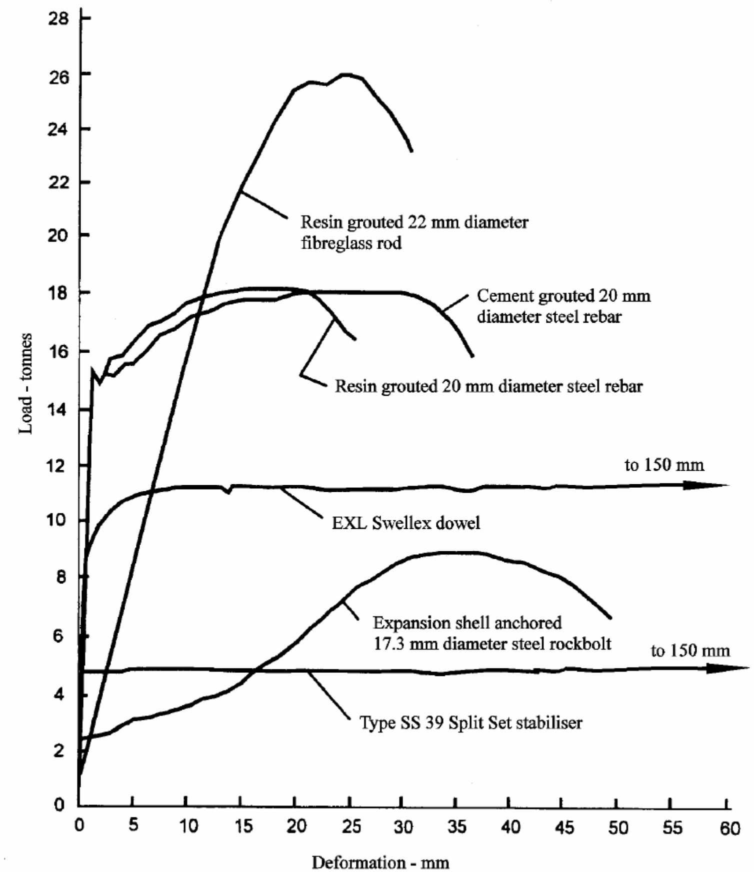


Figure 9: Load-deformation results obtained by Stillborg in tests carried out at Luleå University in Sweden. High strength reinforced concrete with a uniaxial compressive strength of 60 MPa was used for the test blocks and holes were drilled with a percussion rig to simulate in situ rock conditions.

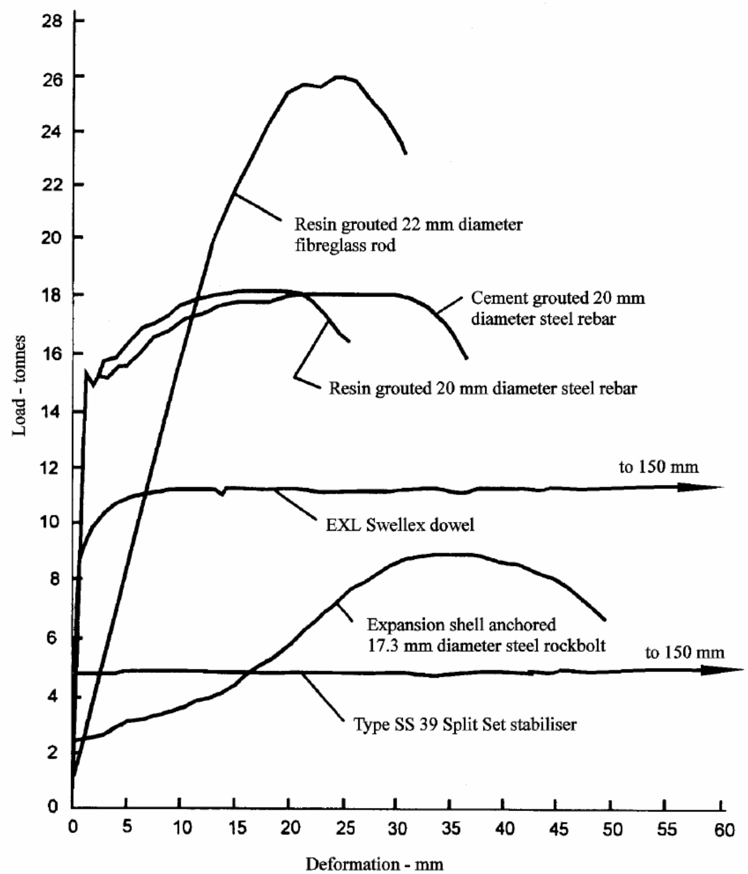


Figure 9: Load-deformation results obtained by Stillborg in tests carried out at Luleå University in Sweden. High strength reinforced concrete with a uniaxial compressive strength of 60 MPa was used for the test blocks and holes were drilled with a percussion rig to simulate in situ rock conditions.

Figure 3: Typical two-component resin cartridge used for anchoring and grouting rockbolts

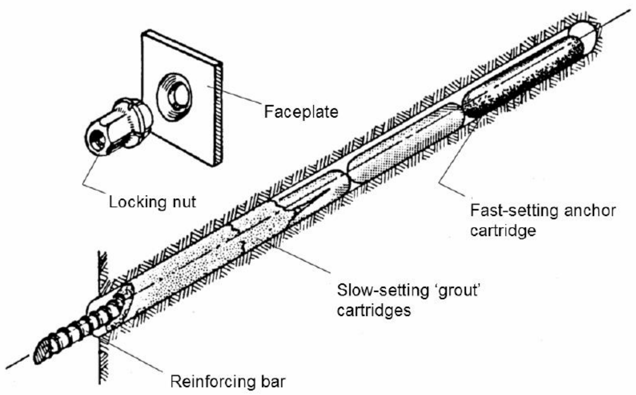


Figure 4: Typical set-up for creating a resin anchored and grouted rockbolt. Resin grouting involves placing slow-setting resin cartridges behind the fast-setting anchor cartridges and spinning the bolt rod through them all to mix the resin and catalyst. The bolt is tensioned after the fast-setting anchor resin has set and the slow-setting resin sets later to grout the rod in place.

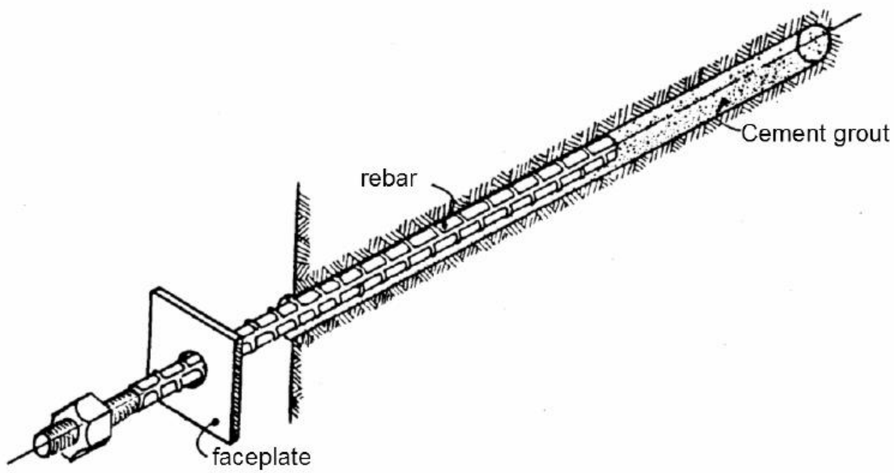


Figure 5: Grouted dowel using a deformed bar inserted into a grout-filled hole

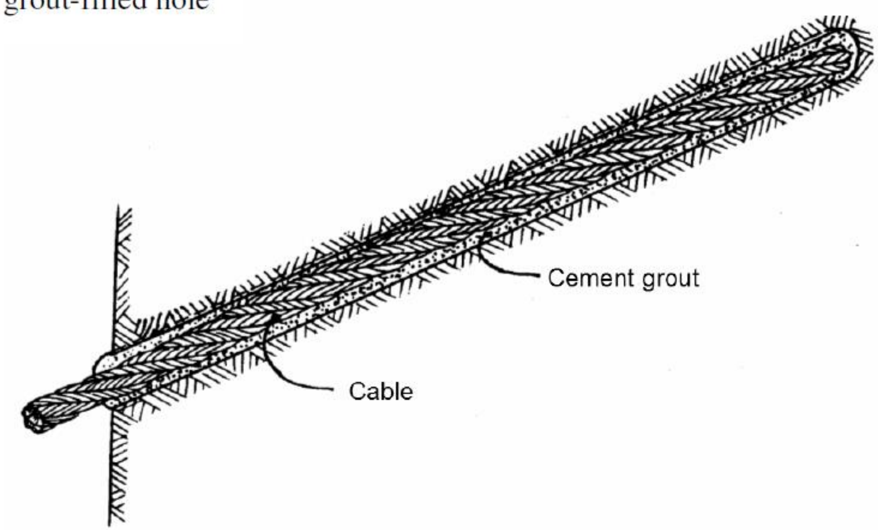


Figure 6: Grouted cables can be used in place of rebar when more flexible support is required or where impact and abrasion can cause problems with rigid support.

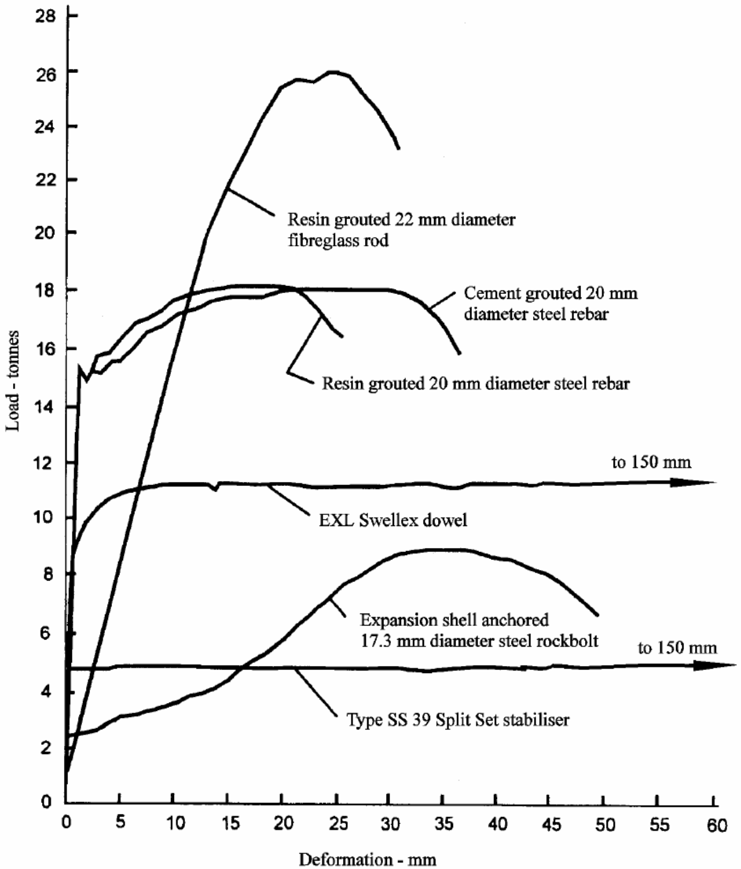


Figure 9: Load-deformation results obtained by Stillborg in tests carried out at Luleå University in Sweden. High strength reinforced concrete with a uniaxial compressive strength of 60 MPa was used for the test blocks and holes were drilled with a percussion rig to simulate in situ rock conditions.

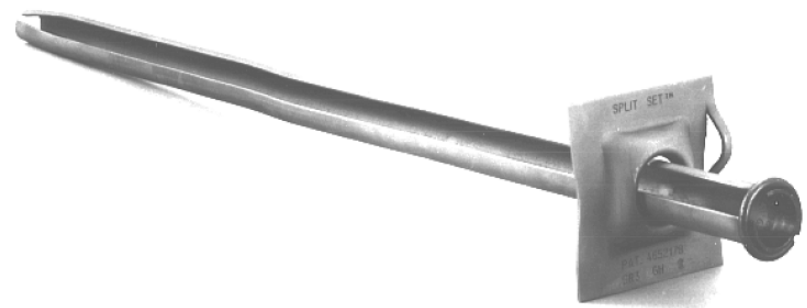


Figure 7: Split Set stabiliser. Ingersol-Rand photograph.

Table 1: Split Set specifications (After Split Set Division, Ingersol-Rand Company).

Split Set stabiliser model	*SS-33*	*SS-39*	*SS-46*
Recommended nominal bit size	31 to 33 mm	35 to 38 mm	41 to 45 mm
Breaking capacity, average	10.9 tonnes	12.7 tonnes	16.3 tonnes
minimum	7.3 tonnes	9.1 tonnes	13.6 tonnes
Recommended initial anchorage (tonnes)	2.7 to 5.4	2.7 to 5.4	4.5 to 8..2
Tube lengths	0.9 to 2.4 m	0.9 to 3.0 m	0.9 to 3.6 m
Nominal outer diameter of tube	33 mm	39 mm	46 mm
Domed plate sizes	150x150 mm	150x150 mm	150x150 mm
	125x125 mm	125x125 mm	
Galvanised system available	yes	yes	yes
Stainless steel model available	no	yes	no

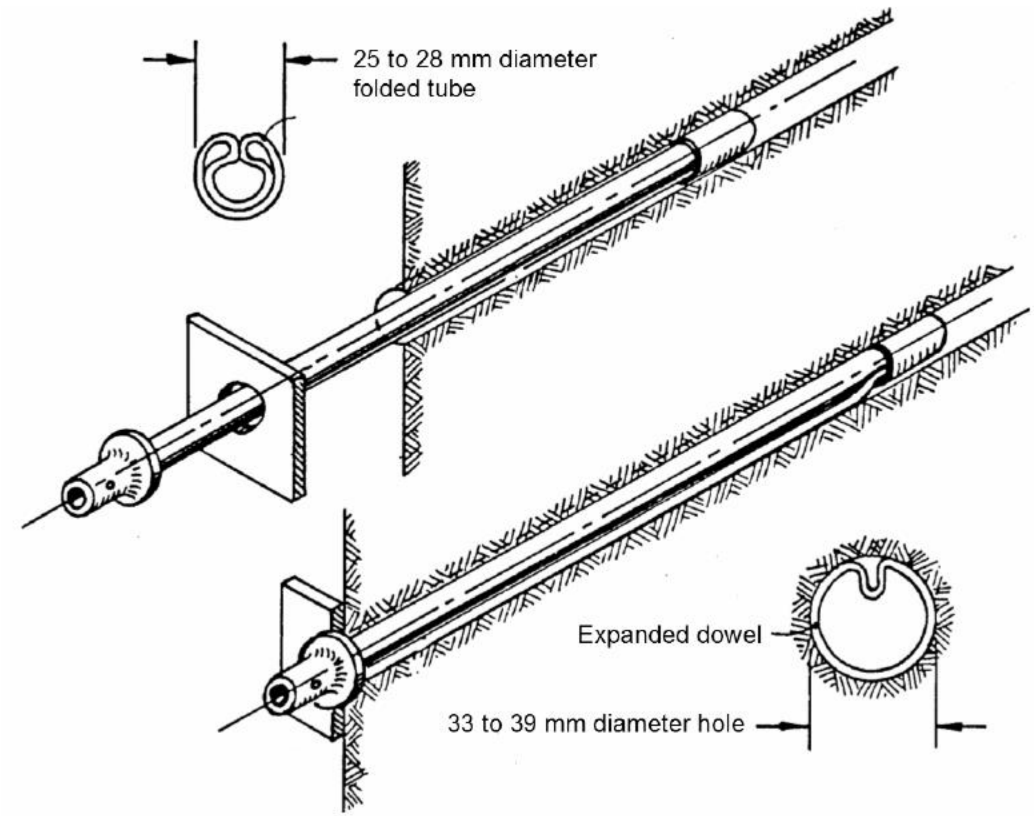


Figure 8: Atlas Copco 'Swellex' dowel.

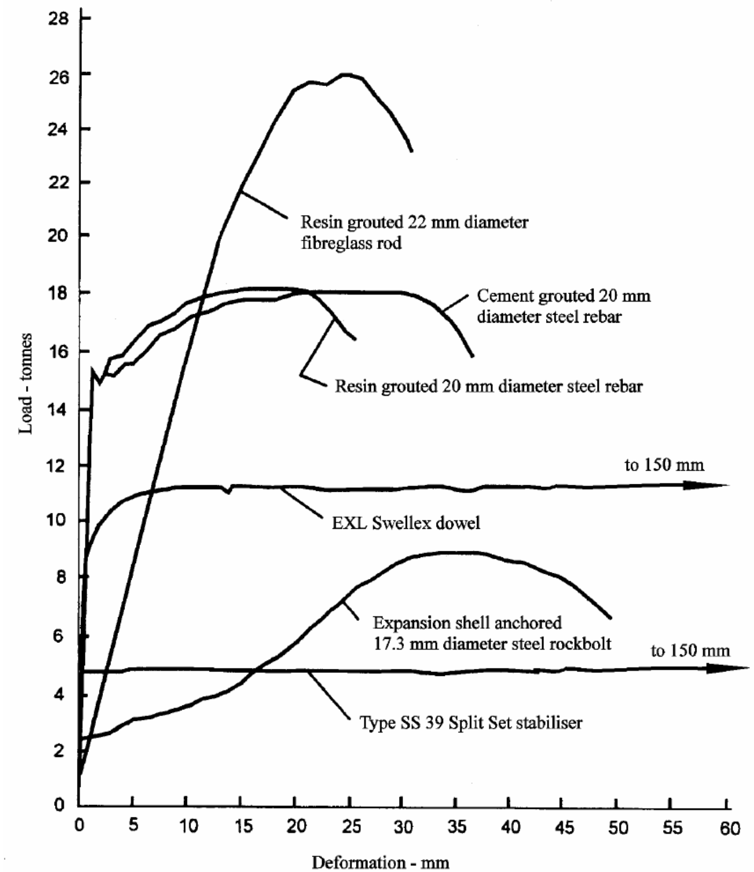


Figure 9: Load-deformation results obtained by Stillborg in tests carried out at Luleå University in Sweden. High strength reinforced concrete with a uniaxial compressive strength of 60 MPa was used for the test blocks and holes were drilled with a percussion rig to simulate in situ rock conditions.

The forces and displacements associated with a stressed cable grouted into a borehole in rock are illustrated in Figure 11.

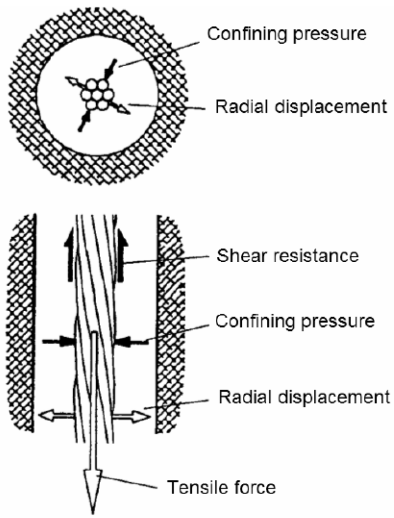


Figure 11: Forces and displacements associated with a stressed cable grouted into a borehole in rock.

As the cable pulls out of the grout, the resultant interference of the spiral steel wires with their associated grout imprints or flutes causes radial displacement or dilation of the interface between the grout and the cable. The radial dilation induces a confining pressure that is proportional to the combined stiffness of the grout and the rock surrounding the borehole. The shear stress, which resists sliding of the cable, is a product of the confining pressure and the coefficient of friction between the steel wires and the grout. Shear strength, therefore, increases with higher grout strength, increases in the grout and the rock stiffness and increases in the confining stresses in the rock after installation of the cable. Conversely, decrease in shear strength can be expected if any of these factors decrease or if the grout crushes.

The results of a comprehensive testing programme on Portland cement grouts have been summarised by Hyett et al (1992) and Figures 12, and 13 are based upon this summary. Figure 12 shows the decrease in both 28 day uniaxial compressive strength and deformation modulus with increasing water/cement ratio. Figure 13 gives Mohr failure envelopes for three water/cement ratios. These results show that the properties of grouts with water/cement ratios of 0.35 to 0.4 are significantly better than those with ratios in excess of 0.5. However, Hyett et al found that the scatter in test results increased markedly for water/cement ratios less than 0.35. The implication is that the ideal water/cement ratio for use with cable reinforcement lies in the range of 0.35 to 0.4.

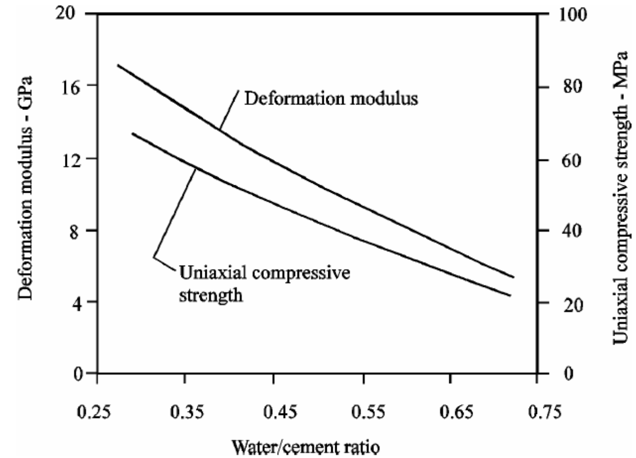
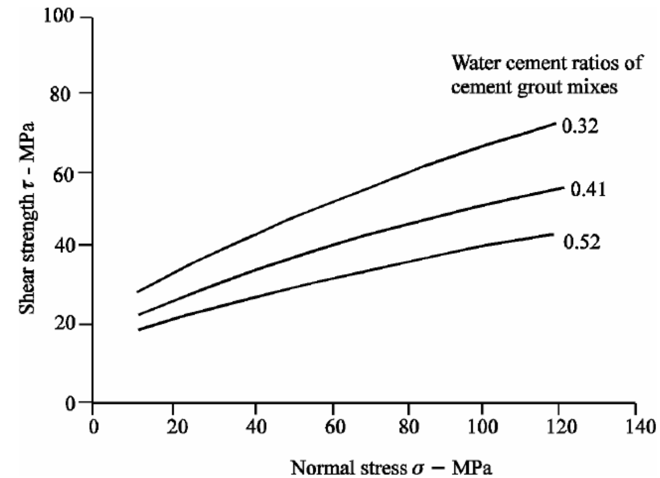


Figure 12: Relationship between the water/cement ratio and the average uniaxial compressive strength and deformation modulus for grouts testes at 28 days.



w/c ratio	σ_c *MPa*	*constant m*	*constant s*	*Friction angle* $\phi°$	*Cohesion c MPa*
0.32	78	3.05	1	24	25
0.41	54	2.14	1	20	19
0.52	38	1.67	1	17	14

Figure 13: Mohr failure envelopes for the peak strength of grouts with different water/cement ratios, tested at 28 days.

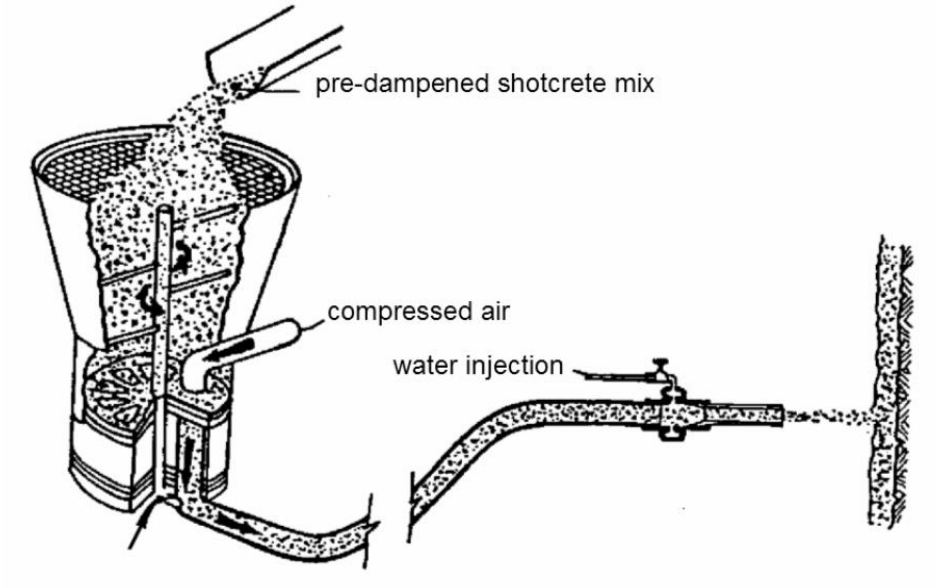


Figure 1: Simplified sketch of a typical dry mix shotcrete system. After Mahar et al (1975).

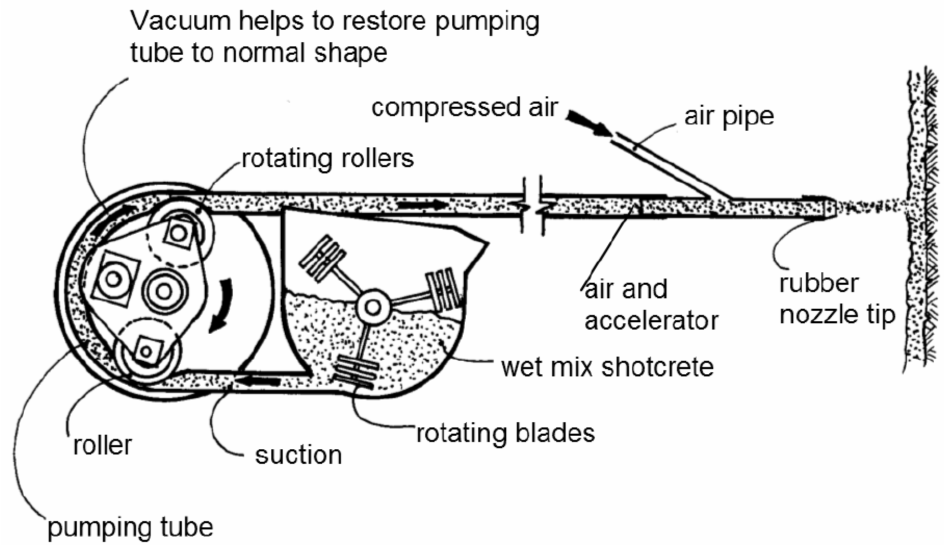


Figure 2: One typical type of wet mix shotcrete machine. After Mahar et al (1975).

Steel fibre reinforced micro silica shotcrete

Of the many developments in shotcrete technology in recent years, two of the most significant were the introduction of silica fume, used as a cementitious admixture, and steel or polypropylene fibre reinforcement.

Silica fume or micro silica is a by-product of the ferro silicon metal industry and is an extremely fine pozzolan. Pozzolans are cementitious materials which react with the calcium hydroxide produced during cement hydration. Silica fume, added in quantities of 8 to 13% by weight of cement, can allow shotcrete to achieve compressive strengths which are double or triple the value of plain shotcrete mixes. The result is an extremely strong, impermeable and durable shotcrete. Other benefits include reduced rebound, improved flexural strength, improved bond with the rock mass and the ability to place layers of up to 200 mm thick in a single pass because of the shotcrete's 'stickiness'. However, when using wet mix shotcrete, this stickiness decreases the workability of the material and superplaticizers are required to restore this workability.

Steel fibre reinforced shotcrete was introduced in the 1970s and has since gained world-wide acceptance as a replacement for traditional wire mesh reinforced plain shotcrete. The main role that reinforcement plays in shotcrete is to impart ductility to an otherwise brittle material. As pointed out earlier, rock support is only called upon to carry significant loads once the rock surrounding an underground excavation deforms. This means that unevenly distributed non-elastic deformations of significant magnitude may overload and lead to failure of the support system, unless that system has sufficient ductility to accommodate these deformations.

Table 1: Typical steel fibre reinforced silica fume shotcrete mix designs (After Wood, 1992)

Components	Dry mix		Wet mix	
	kg./m^3	% dry materials	kg./m^3	% wet materials
Cement	420	19.0	420	18.1
Silica fume additive	50	2.2	40	1.7
Blended aggregate	1,670	75.5	1,600	68.9
Steel fibres	60	2.7	60	2.6
Accelerator	13	0.6	13	0.6
Superplasticizer	-	-	6 litres	0.3
Water reducer	-	-	2 litres	0.1
Air entraining admixture	-	-	if required	
Water	controlled at nozzle		180	7.7
Total	2,213	100	2,321	100

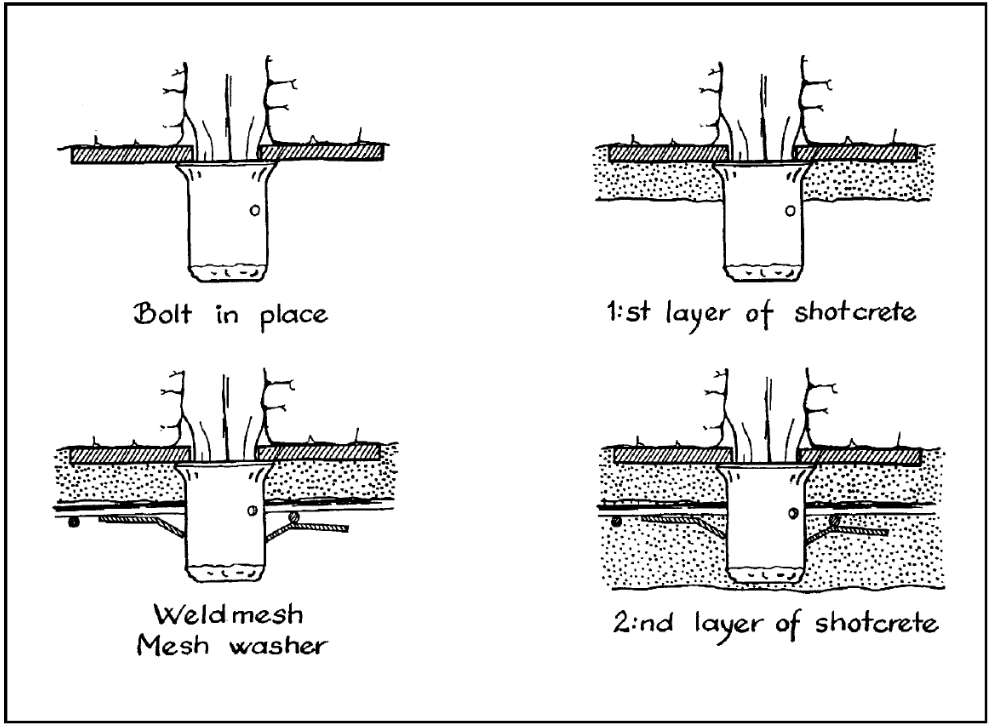


Figure 5-15. Dowels with end hardware embedded in shotcrete

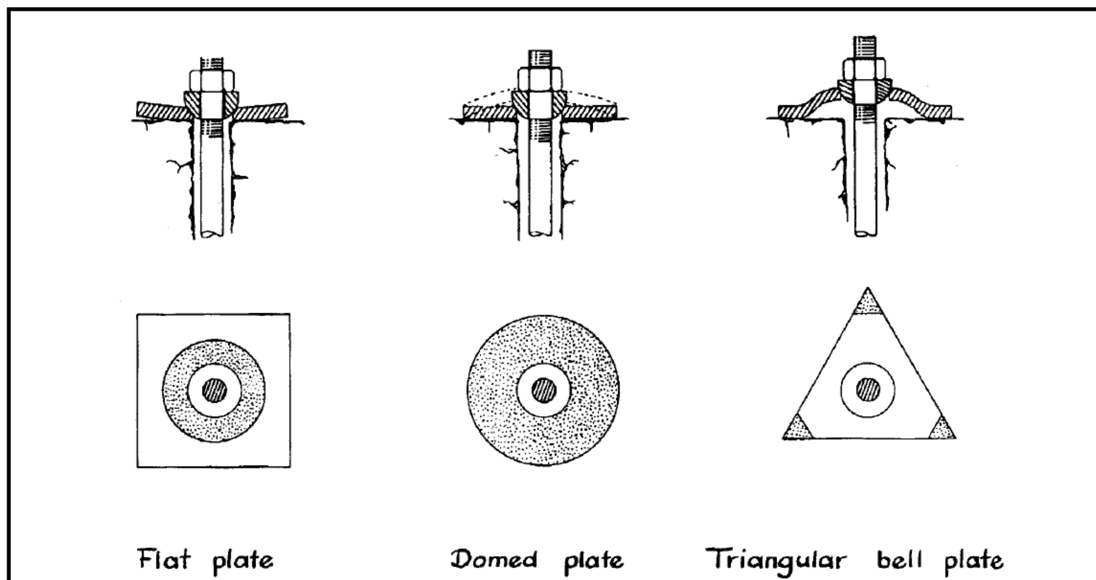


Figure 5-16. Different types of end hardware

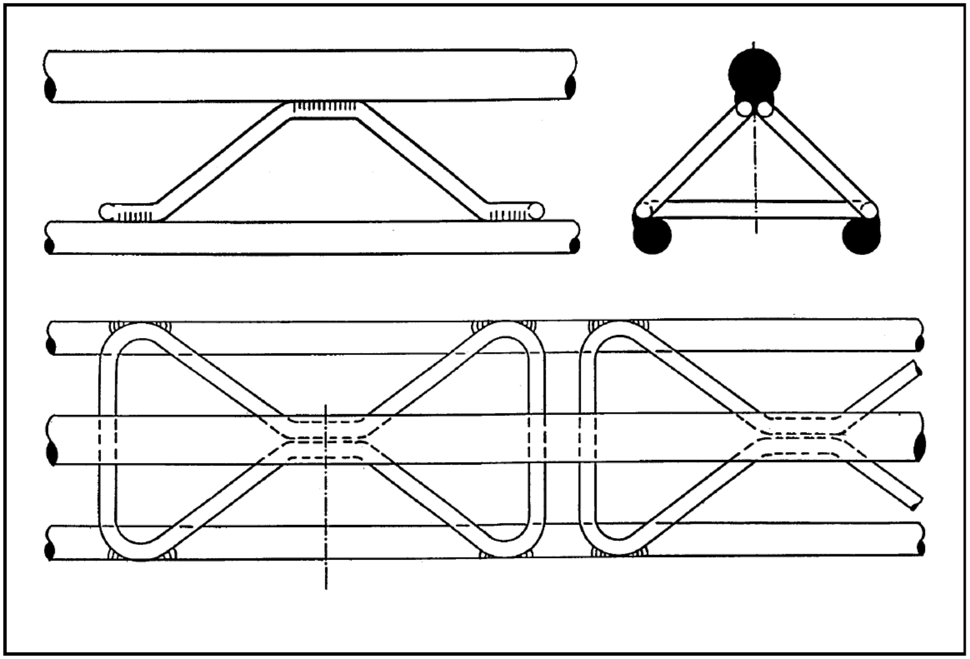


Figure 5-18. Lattice girders

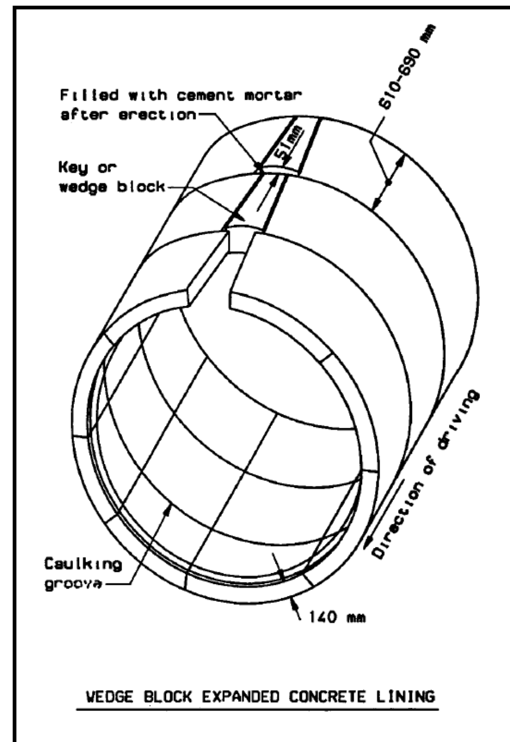


Figure 5-22. Wedge block expanded concrete lining

Figure 3: Bagged pre-mixed dry shotcrete components being delivered into a hopper feeding a screw conveyor, fitted with a pre-dampener, which discharges into the hopper of a shotcrete machine

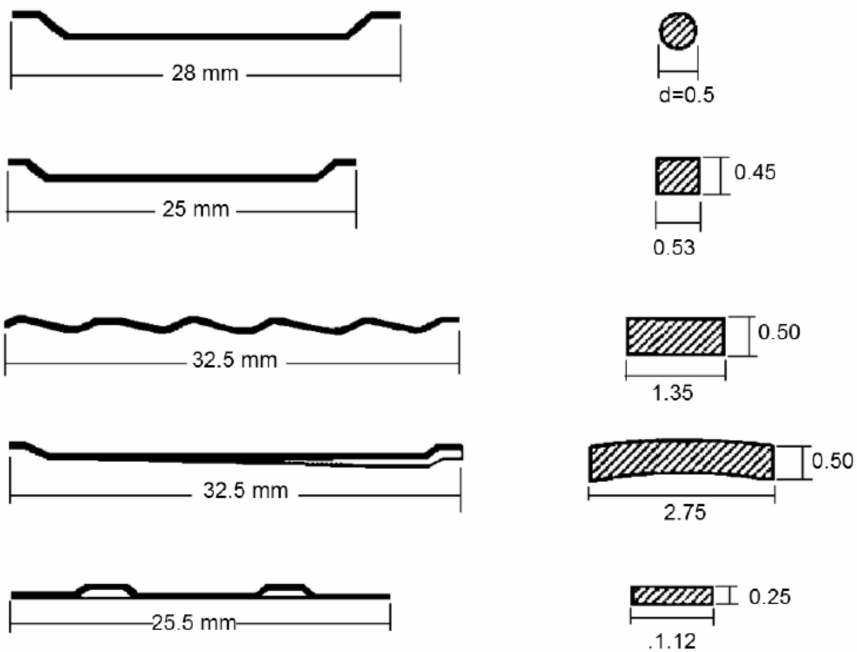


Figure 4. Steel fibre types available on the North American market. After Wood et al (1993). (Note: all dimensions are in mm).

Wood et al (1993) have reported the results of a comprehensive comparative study in which all of the fibres shown in Figure 4 were used to reinforce shotcrete samples which were then subjected to a range of tests. Plain and fibre reinforced silica fume shotcrete samples were prepared by shooting onto vertical panels, using both wet and dry mix processes. The fibre reinforced samples all contained the same steel fibre dosage of 60 kg/m^3 (see Table 1). All the samples were cured under controlled relative humidity conditions and all were tested seven days after shooting.

These tests showed that the addition of steel fibres to silica fume shotcrete enhances both the compressive and flexural strength of the hardened shotcrete by up to 20%. A significant increase in ductility was also obtained in all the tests on fibre reinforced samples, compared with plain samples. While different fibres gave different degrees of improvement, all of the fibres tested were found to exceed the levels of performance commonly specified in North America (i.e. 7-day compressive strength of 30 MPa for dry mix, 25 MPa for wet mix and 7-day flexural strength of 4 MPa).

Kompen (1989) carried out bending tests on slabs of unreinforced shotcrete and shotcrete reinforced with 'Dramix'[1] steel fibres, shown in Figure 5. The shotcrete had an unconfined compressive strength, determined from tests on cubes, of 50 MPa. The results of these tests are reproduced in Figure 6. The peak strength of these slabs increased by approximately 85% and 185% for 1.0 and 1.5 volume % of fibres, respectively. The ductility of the fibre reinforced slabs increased by approximately 20 and 30 times for the 1.0 and 1.5 volume % of fibres, respectively.

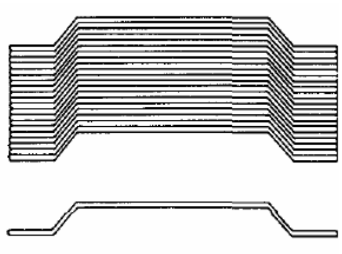


Figure 5: 'Dramix' steel fibres used in slab bending tests by Kompen (1989). The fibres are glued together in bundles with a water soluble glue to facilitate handling and homogeneous distribution of the fibres in the shotcrete.

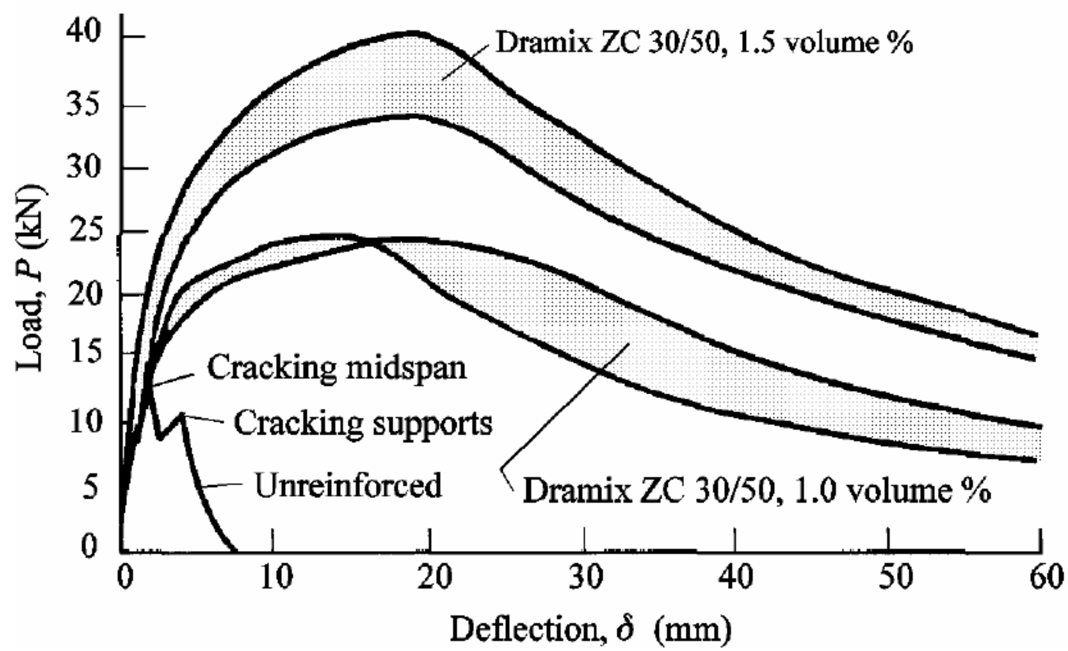


Figure 6: Load deflection curves for unreinforced and steel fibre reinforced shotcrete slabs tested in bending. After Kompen (1989).

While steel fibre reinforced shotcrete has been widely accepted in both civil and mining engineering, mesh reinforced shotcrete is still widely used and is preferred in some applications. In very poor quality, loose rock masses, where adhesion of the shotcrete to the rock surface is poor, the mesh provides a significant amount of reinforcement, even without shotcrete. Therefore, when stabilising slopes in very poor quality rock masses or when building bulkheads for underground fill, weldmesh is frequently used to stabilise the surface or to provide reinforcement. In such cases, plain shotcrete is applied later to provide additional support and to protect the mesh against corrosion.

Figure 7: Chainlink mesh, while very strong and flexible, is not ideal for shotcrete application because it is difficult for the shotcrete to penetrate the mesh.

Chainlink mesh, used in many underground mining excavations to support loose rock, is not usually suitable for shotcrete reinforcement. This is because penetration of the shotcrete is inhibited by the twisted joints as illustrated in Figure 7. This allows air cavities to form behind the mesh and these may allow water to enter and cause corrosion of the mesh.

On the other hand, weldmesh, tightly pinned against the rock face as illustrated in Figure 8, is generally ideal for shotcrete applications. Typically the weldmesh should be made from 4 mm diameter wire welded into a 100 mm x 100 mm grid. This type of mesh is strong enough for most underground applications and the sheets are light enough to he handled by one man.

Figure 8: Welded wire mesh, firmly attached to the rock surface, provides excellent reinforcement for shotcrete.

A well-trained operator can produce excellent quality shotcrete manually, when the work area is well-lit and well-ventilated, and when the crew members are in good communication with each other using prescribed hand signals or voice activated FM radio headsets. However, this is a very tiring and uncomfortable job, especially for overhead shooting, and compact robotic systems are increasingly being used to permit the operator to control the nozzle remotely. Typical robotic spray booms are illustrated in Figures 9, 10 and 11.

Figure 9: A truck mounted shotcrete robot being used in a large civil engineering tunnel. Note that the distance between the nozzle and the rock surface is approximately one metre.

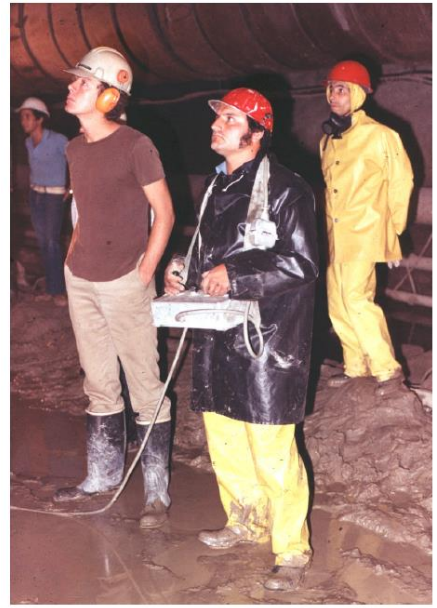


Figure 11: Shotcrete operator using a remotely controlled unit to apply shotcrete to a rock face in a large civil engineering excavation.

Figure 12: Plastic pipes used to provide drainage for a shotcrete layer applied to a rock mass with water-bearing joints.