UNIT-I
MCE-164
Geotechnics of Hill Area

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Topics

1. INTRODUCTION
2. GRAVITY AND CANTILEVER WALLS
3. PROPORTIONING RETAINING WALLS
4. APPLICATION OF LATERAL EARTH PRESSURE THEORIES TO DESIGN
5. STABILITY CHECKS
   - Check for Overturning
   - Check for Sliding Along the Base
   - Check for Bearing Capacity Failure
   - Example
   - Factor of Safety Against Overturning
   - Factor of Safety Against Sliding
   - Factor of Safety Against Bearing Capacity Failure

INTRODUCTION

- In general, retaining walls can be divided into two major categories: (a) conventional retaining walls, and

(b) mechanically stabilized earth walls.

- Conventional retaining walls can generally be classified as
  1. Gravity retaining walls
  2. Semi-gravity retaining walls
  3. Cantilever retaining walls
  4. Counterfort retaining walls

- *Gravity retaining walls* are constructed with plain concrete or stone masonry. They depend on their own weight and any soil resting on the masonry for stability. This type of construction is not economical for high walls.

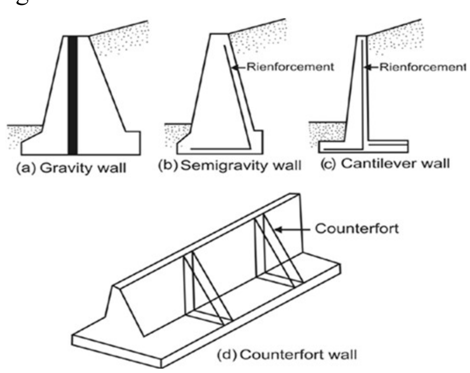


Figure 1 Types of retaining wall

- In many cases, a small amount of steel may be used for the construction of gravity walls, thereby minimizing the size of wall sections. Such walls are generally referred to as semi-gravity walls

- ❖ Cantilever retaining walls are made of reinforced concrete that consists of a thin stem and a base slab. This type of wall is economical to a height of about 25 ft (8 m).
- ❖ Counterfort retaining walls are similar to cantilever walls. At regular intervals, however, they have thin vertical concrete slabs known as counterforts that tie the wall and the base slab together. The purpose of the counterforts is to reduce the shear and the bending moments.

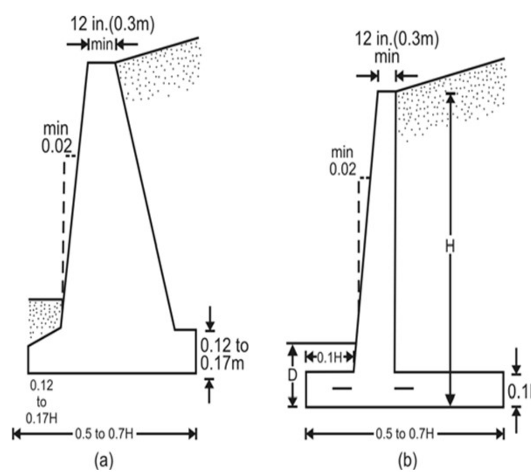
- To design retaining walls properly, an engineer must know the basic soil parameters-that is, the unit weight, angle of friction, and cohesion-for the soil retained behind the wall and the soil below the base slab. Knowing the properties of the soil behind the wall enables the engineer to determine the lateral pressure distribution that has to be designed for.

There are two phases in the design of conventional retaining walls.

- First, with the lateral earth pressure known, the structure as a whole is checked for stability. That includes checking for possible overturning, sliding, and bearing capacity failures.
- Second, each component of the structure is checked for adequate strength, and the steel reinforcement of each component is determined.
- ✓ Mechanically stabilized retaining walls have their backfills stabilized by inclusion of reinforcing elements such as metal strips, bars, welded wire mats, geotextiles, and geogrids. These walls are relatively flexible and can sustain large horizontal and vertical displacement without much damage.

PROPORTIONING RETAINING WALLS

- When designing retaining walls, an engineer must assume some of the dimensions, called proportioning, which allows the engineer to check trial sections for stability. If the stability checks yield undesirable results, the sections can be changed and rechecked. Figure shows the general proportions of various retaining walls components that can be used for initial checks.
- Note that the top of the stem of any retaining wall should not be less than about 12 in. (≈0.3 m) for proper placement of concrete. The depth, D, to the bottom of the base slab should be a minimum of 2 ft (≈ 0.6 m). However, the bottom of the base slab should be positioned below the seasonal frost line.
- For counterfort retaining walls, the general proportion of the stem and the base slab is the same as for cantilever walls. However, the counterfort slabs may be about 12 in. (≈0.3 m) thick and spaced at center-to-center distances of $0.3H$ to $0.7H$.



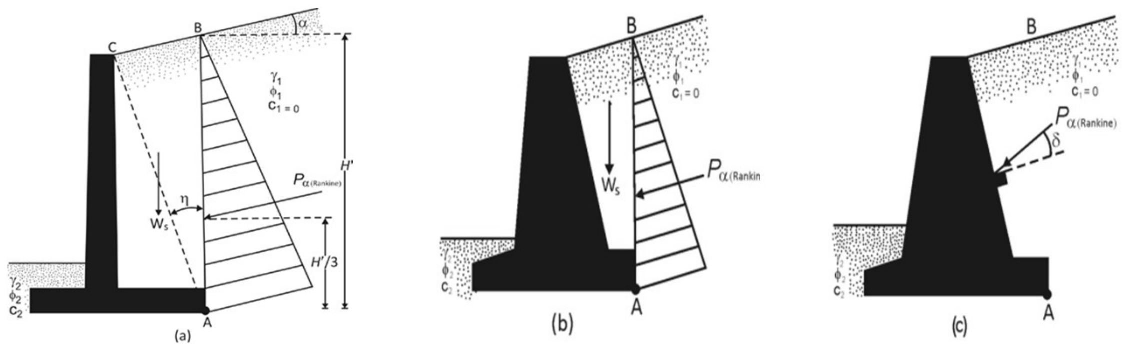
Approximate dimensions for various components of retaining wall for initial stability checks: (a) gravity wall; (b) cantilever wall [note: minimum dimension of D is 2 ft (≈ 0.6 m)]

APPLICATION OF LATERAL EARTH PRESSURE THEORIES TO DESIGN

- To use these theories in design, an engineer must make several simple assumptions. In the case of cantilever walls, use of the Rankine earth pressure theory for stability checks involves drawing a vertical line *AB* through point *A*. (which is located at the edge of the heel of the base slab. The Rankine active condition is assumed to exist along the vertical plane *AB*. Rankine active earth pressure equations may then be used to calculate the lateral pressure on the face *AB*. In the analysis of stability for the wall, the force $P_{\alpha(\text{Rankine})}$, the weight of soil above the heel, *Ws*, and the weight of the concrete, W_c, all should be taken into consideration. The assumption for the development of Rankine active pressure along the soil face *AB* is theoretically concrete if the shear zone bounded by the line *AB* is not obstructed by the stem of the wall. The angle, η, that the line *AB* makes with the vertical is

$$\eta = 45 + \frac{\alpha}{2} - \frac{\phi}{2} - \sin^{-1}\left(\frac{\sin\alpha}{\sin\phi}\right)$$

- A similar type of analysis may be used for gravity walls. However, Coulomb's theory also may be used. If *Coulomb's active pressure theory* is used, the only forces to be considered are *Pa (Coulomb)* and the weight of the wall, W_c.



Assumption for the determination of lateral earth pressure: (a) cantilever wall; (b) and (c) gravity wall

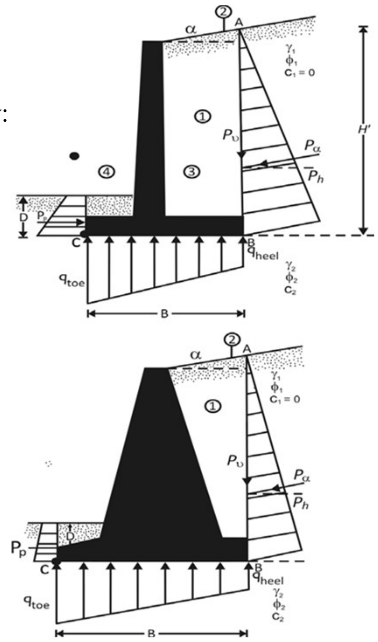
- If Coulomb's earth pressure theory is used, it will be necessary to know the range of the wall friction angle δ with various types of backfill material. Following are some ranges of wall friction angle for masonry or mass concrete walls:

Backfill material	Range of δ (deg)
Gravel	27-30
Coarse sand	20-28
Fine sand	15-25
Stiff clay	15-20

STABILITY CHECKS

To check the stability of a retaining wall, the following steps are necessary:

1. Check for *overturning* about its toe
2. Check for *sliding* along its base
3. Check for *bearing capacity failure* of the base
4. Check for *settlement*
5. Check for *overall stability*



Check for overturning: assume that Rankine pressure is valid

➢ Check for Overturning

Figure 1.6 shows the forces acting on a cantilever and a gravity retaining wall, based on the assumption that the Rankine active pressure is acting along a vertical plane *AB* drawn through the heel. *Pp* is the Rankine passive pressure; recall that its magnitude is

$$P_p = \tfrac{1}{2}K_p\gamma_2 D^2 + 2c_2\sqrt{K_p}D$$

Table 1 types of Backfill for Retaining Walls

1. Coarse-grained soil without admixture of fine soil particles, very permeable (clean sand or gravel).

2. Coarse-grained soil of low permeability due to admixture of particles of silt size.

3. Residual soil with stones, fine silty sand, and granular materials with conspicuous clay content.

4. Very soft or soft clay, organic silts, or silty clay.

5. Medium or stiff clay, deposited in chunks and protected in such a way that a negligible amount of water enters the spaces between the chunks during floods or heavy rains. If this condition of protection cannot be satisfied, the clay should not be used as backfill material. With increasing stiffness of the clay, danger to the wall due to infiltration of water increases rapidly.

Where

γ_2 = unit weight of soil in front of the heel and under the base slab

K_p = Rankine passive earth pressure coefficient = $\tan^2(45 + \phi_2/2)$

c_2, ϕ_2 = cohesion and soil friction angle, respectively

The factor of safety against overturning about the toe-that is, about point C in figure 1.6- may be expressed as

$$FS_{(\text{overturning})} = \frac{\Sigma M_R}{\Sigma M_O}$$

Where

ΣM_O = sum of the moments of forces tending to overturn about point C

ΣM_R = sum of the moments of forces tending to resist overturning about point C

The overturning moment is

$$\Sigma M_O = P_h \left(\frac{H'}{3}\right)$$

Where $P_h = P_a \cos \alpha$

For calculation of the resisting moment, ΣM_R (neglecting P_P), a table (such as table 2) can he prepared. The weight of the soil above the heel and the weight of the concrete (or masonry) are both forces that contribute to the resisting moment. Note that the force Pv also contributes to the resisting moment. Pv is the vertical component of the active force Pa, or

$$P_v = P_a \sin \alpha$$

The moment of the force P_v about C is

$$M_v = P_v B = P_a \sin \alpha \, B$$

Where

B = width of the base slab

Table 2 Procedure for Calculation of ΣM_R

Section (1)	Area (2)	Weight/unit length of wall (3)	Moment arm measured from C (4)	Moment about C (5)
1	A_1	$W_1 = \gamma_1 \times A_1$	X_1	M_1
2	A_2	$W_2 = \gamma_2 \times A_2$	X_2	M_2
3	A_3	$W_3 = \gamma_c \times A_3$	X_3	M_3
4	A_4	$W_4 = \gamma_c \times A_4$	X_4	M_4
5	A_5	$W_5 = \gamma_c \times A_5$	X_5	M_5
6	A_6	$W_6 = \gamma_c \times A_6$	X_6	M_6
		P_v	B	M_v
		ΣV		ΣM_R

Note: γ_1 = unit weight of backfill

γ_2 = unit weight of concrete

Once ΣM_R is known, the factor of safety can be calculated as

$$FS_{(\text{overturning})} = \frac{M_1+M_2+M_3+M_4+M_5+M_6}{P_a \cos \alpha (H'/3) - M_v}$$

➢ Check for Sliding Along the Base

The factor of safety against sliding may be expressed by the equation $FS_{(\text{sliding})} = \frac{\Sigma F_R}{\Sigma F_d}$ (1)

Where

ΣF_R = sum of the horizontal resisting forces

ΣF_d = sum of the horizontal driving forces

Figure 1.7 indicates that the shear strength of the soil immediately below the base slab may be represented as

$$s = \sigma \tan \delta + c_a$$

Where

δ = angle of friction between the soil and the base slab

c_a = adhesion between the soil and the base slab

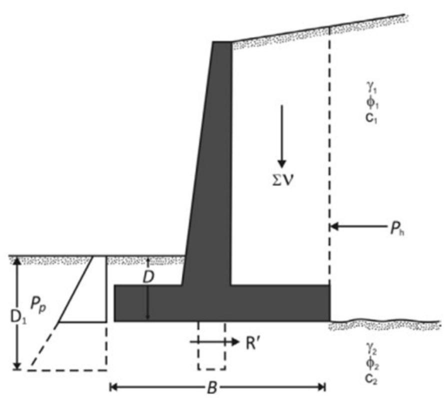


Figure 1.7 Check for sliding along the base

Thus the maximum resisting force that can be derived from the soil per unit length of the wall along the bottom of the base slab is

$$R' = s(\text{area of cross section}) = s(B \times 1) = B\sigma \tan \delta + Bc_a$$

However,

$B\sigma$ = sum of the vertical force = ΣV (see table 2)

So

$$R' = (\Sigma V)\tan\delta + Bc_a + P_p \qquad (2)$$

The only horizontal force that will tend to cause the wall to slide (driving force) is the horizontal component of the active force Pa, so

$$\Sigma F_d = P_a \cos\alpha \qquad (3)$$

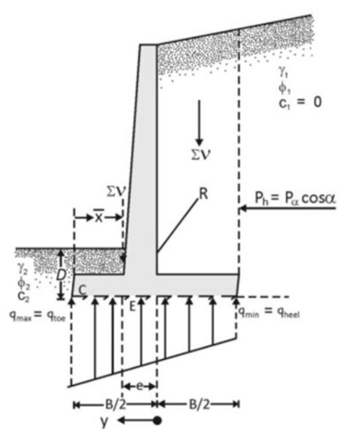
Combings equations (1, 2, and 3) yields

$$FS_{(\text{sliding})} = \frac{(\Sigma V)\tan\delta + Bc_a + P_p}{P_a \cos\alpha}$$

A minimum factor of safety of 1.5 against sliding is generally required.

➢ Check for Bearing Capacity Failure

The vertical pressure as transmitted to the soil by the base slab of the retaining wall should be checked against the ultimate bearing capacity of the soil. The nature of variation of the vertical pressure transmitted by the base slab into the soil. Note that q_{toe} and q_{heel} are the maximum and the minimum pressures occurring at the ends of the toe and heel sections, respectively. The magnitudes of q_{toe} and q_{heel} can be determined in the following manner.



Check for bearing capacity failure

The sum of the vertical forces acting on the base slab is ΣV (see column 3, table 2), and the horizontal force is $P_a \cos\alpha$. Let R be the resultant force, or

$$\vec{R} = \vec{\Sigma V} + \vec{(P_a \cos\alpha)}$$

The net moment of these forces about point C

$$M_{\text{net}} = \Sigma M_R - \Sigma M_O$$

Note that the values of ΣM_R and ΣM_O have been previously determined (see column 5, table 2). Let the line of action of the resultant, R, interest the base slab at E. The distance C then is

$$\overline{CE} = \bar{X} = \frac{M_{\text{net}}}{\Sigma V}$$

Hence the eccentricity of the resultant, R, may be expressed as

$$e = \frac{B}{2} - \overline{CE}$$

The pressure distribution under the base slab may be determined by using the simple principles of mechanics of materials: $q = \frac{\Sigma V}{A} \pm \frac{M_{\text{net}}\, y}{I}$ (4)

Where

M_{net} = moment = $(\Sigma V)e$

I = moment of inertia per unit length of the base section $= \frac{1}{12}(1)(B^2)$

For maximum and minimum pressures, the value of y in equation (4) equals $BB/2$. Substituting the preceding values into equation (4) gives

$$q_{\max} = q_{\text{toe}} = \frac{\Sigma V}{(B)(1)} + \frac{e(\Sigma V)\frac{B}{2}}{\left(\frac{1}{12}\right)(B^2)} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right) \quad (5)$$

Similarly,

$$q_{\min} = q_{\text{heel}} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right) \quad (6)$$

Note that ΣVV includes the soil weight, as shown in table 2, and that, when the value of the eccentricity, e, becomes greater than $B/6$, qqmin becomes negative [equation (6)]. Thus, there will be some tensile stress at the end of the heel section. This stress is not desirable because the tensile strength of soil is very small. If the analysis of a design shows that $e > B/6$, the design should be reproportioned and calculations redone.

The relationships for the ultimate bearing capacity of a shallow foundation is

$$q_u = c_2 N_c F_{cd} F_{ci} + q N_q F_{qd} F_{qi} + \frac{1}{2}\gamma_2 B' N_\gamma F_{\gamma d} F_{\gamma i} \quad (7)$$

Where

$$q = \gamma_2 D$$

$$B' = B - 2e$$

$$F_{cd} = 1 + 0.4\frac{D}{B'}$$

$$F_{qd} = 1 + 2\tan\phi_2(1 - \sin\phi_2)^2\frac{D}{B'}$$

$$F_{\gamma d} = 1$$

$$F_{ci} = F_{qi} = \left(1 - \frac{\psi^\circ}{90^\circ}\right)^2$$

$$F_{\gamma i} = \left(1 - \frac{\psi^\circ}{\phi_2^\circ}\right)^2$$

$$\psi^\circ = \tan^{-1}\left(\frac{P_a\cos\alpha}{\Sigma V}\right)$$

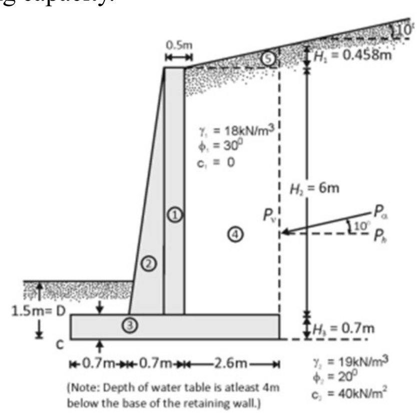
Once the ultimate bearing capacity of the soil has been calculated by using equation (7), the factor of safety against bearing capacity failure can be determined.

$$FS_{(\text{bearing capacity})} = \frac{q_u}{q_{\max}} \tag{8}$$

Generally, a factor of safety of 3 is required. In the case of retaining walls, the width B is large. Hence the ultimate load q_u will occur at a fairly large foundation settlement. A factor of safety of 3 against bearing capacity failure may not ensure, in all cases, that settlement of the structure will be within the tolerable limit. Thus this situation needs further investigation.

Example 1

The cross section of a cantilever retaining wall is shown below. Calculate the factors of safety with respect to overturning and sliding and bearing capacity.



$H' = H_1 + H_2 + H_3 = 2.6 \tan 10^{\circ} + 6 + 0.7 = 0.458 + 6 + 0.7 = 7.158 \text{ m}$

The Rankine active force per unit length of wall= $P_p = \frac{1}{2}\gamma_1 H'^2 K_a$. For $\phi_1 = 30^{\circ}, \alpha = 10^{\circ}, K_a$ is equal to 0.350 (table 2 from chapter 6). Thus,

$P_a = \frac{1}{2}(18)(7.158)^2(0.35) = 161.4 \text{ kN/m}$

$P_v = P_a \sin 10^{\circ} = 161.4(\sin 10^{\circ}) = 28.03 \text{ kN/m}$

$P_h = P_a \cos 10^{\circ} = 161.4(\cos 10^{\circ}) = 158.95 \text{ kN/m}$

Factor of Safety Against Overturning

The following table can now be prepared for determination of the resisting moment:

Section no.	Area (m^2)	Weight/unit length (kN/m)	Moment from C (kN/m)	Moment (kN − m)
1	$6 \times 0.5 = 3$	70.74	1.15	81.35
2	$\frac{1}{2}(0.2)6 = 0.6$	14.15	0.833	11.79
3	$4 \times 0.7 = 2.8$	66.02	2.0	132.04
4	$6 \times 2.6 = 15.6$	280.80	2.7	758.16
5	$\frac{1}{2}(2.6)(0.458) = 0.595$	10.71	3.13	33.52
	$P_v =$ 28.03	4.0		112.12
	Σ 1128.98	$\Sigma V = 470.45$		$= \Sigma M_R$
For section numbers, refer to figure 7.11,				
		$\gamma_{concrete} = 2358 \text{ kN/m}^3$		

The overturning moment, M_O

$M_O = P_h\left(\frac{H'}{3}\right) = 158.95\left(\frac{7.158}{3}\right) = 379.25 \text{ kN} - \text{m}$

$FS_{(\text{overturning})} = \frac{\Sigma M_R}{M_O} = \frac{1128.98}{379.25} = 2.98 > 2 - \text{OK}$

Factor of Safety Against Sliding

From equation (11)

$FS_{(\text{sliding})} = \frac{(\Sigma V)\tan(k_1\phi_2) + Bk_2c_2 + P_p}{P_a \cos\alpha}$

Let $k_1 = k_2 = \frac{2}{3}$

Also

$P_p = \frac{1}{2}K_p\gamma_2 D^2 + 2c_2\sqrt{K_p}D$

$K_p = \tan^2\left(45 + \frac{\phi_2}{2} = \tan^2(45 + 10) = 2.04\right)$

$D = 1.5 \text{ m}$

So

$P_p = \frac{1}{2}(2.04)(19)(1.5)^2 + 2(40)(\sqrt{2/04})\ (1.5)$

$= 43.61 + 171.39 = 215 \text{ kN/m}$

Hence

$FS_{(\text{sliding})} = \frac{(470.45)\tan\left(\frac{2\times 20}{3}\right) + (4)\left(\frac{2}{3}\right)(40) + 215}{158.95}$

$= \frac{111.5 + 106.67 + 215}{158.95} = 2.73 > 1.5 - \text{OK}$

Note: For some designs, the depth D for passive pressure calculation may be taken to be *equal to the thickness of the base slab.*

Factor of Safety Against Bearing Capacity Failure

Combining equations (16, 17 and 18),

$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_O}{\Sigma V} = \frac{4}{2} - \frac{1128.98 - 379.25}{470.45}$

$= 0.406\text{m} < \frac{B}{6} = \frac{4}{6} = 0.666 \text{ m}$

$$q_{heel}^{toe} = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{470.45}{4}\left(1 \pm \frac{6\times0.406}{4}\right) = \begin{matrix}189.2\ \text{kN/m}^2\ \text{(toe)}\\ 45.9\ \text{kN/m}^2\text{(heel)}\end{matrix}$$

The ultimate bearing capacity of the soil can be determined from equation (22):

$$q_u = c_2 N_c F_{cd} F_{ci} + q N_q F_{qd} F_{qi} + \tfrac{1}{2}\gamma_2 B' N_\gamma F_{\gamma d} F_{\gamma i}$$

For $\phi_2 = 20^\circ$ (table 4 from chapter 3), $N_c = 14.83, N_q = 6.4$ and $N_\gamma = 5.39$. Also

$$q = \gamma_2 D = (19)(1.5) = 28.5\ \text{kN/m}^2$$

$$B' = B - 2e = 4 - 2(0.406) = 3.188\ \text{m}$$

$$F_{cd} = 1 + 0.4\left(\frac{D}{B'}\right) = 1 + 0.4\left(\frac{1.5}{3.188}\right) = 1.188$$

$$F_{qd} = 1 + 2\tan\phi_2(1-\sin\phi_2)^2\left(\frac{D}{B'}\right) = 1 + 0.315\left(\frac{1.5}{3.188}\right) = 1.148$$

$$F_{\gamma d} = 1$$

$$F_{ci} = F_{qi} = \left(1 - \frac{\psi^\circ}{90^\circ}\right)^2$$

$$\psi = \tan^{-1}\left(\frac{P_a \cos\alpha}{\Sigma V}\right) = \tan^{-1}\left(\frac{158.95}{470.45}\right) = 18.67^\circ$$

So

$$F_{ci} = F_{qi} = \left(1 - \frac{18.67^\circ}{90}\right) = 0.628$$

$$F_{\gamma i} = \left(1 - \frac{\psi}{\phi}\right)^2 = \left(1 - \frac{18.67}{20}\right) \approx 0$$

Hence

$$q_u = (40)(14.83)(1.188)(0.628) + (28.5)(6.4)(1.148)(0.628) + \tfrac{1}{2}(19)(5.93)(3.188)(1)(0)$$

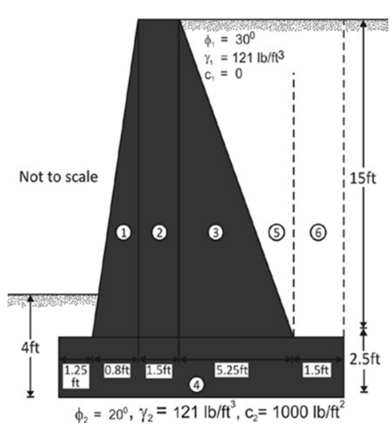
$$= 442.57 + 131.50 + 0 = 574.07\ \text{kN/m}^2$$

$$FS_{(\text{bearing capacity})} = \frac{q_u}{q_{toe}} = \frac{574.07}{189.2} = 3.03 > 3 - \text{OK}$$

Example 2

A concrete gravity retaining wall is. Determine

a. The factor of safety against overturningshown in below
b. The factor of safety against sliding
c. The pressure on the soil at the toe and heel (Note: Unit weight of concrete = $\gamma\gamma_{\alpha}$= 150 lb/ft^3).



Solution

$H' = 15 + 2.5 = 17.5$ ft

$K_a = \tan^2\left(45 - \frac{\phi_1}{2}\right) = \tan^2\left(45 - \frac{30}{2}\right) = \frac{1}{3}$

$P_a = \frac{1}{2}\gamma(H')^2 K_a = \frac{1}{2}(121)(17.5)^2\left(\frac{1}{3}\right) = 6176$ lb/ft

$= 6.716$ kip/ft

Since $\alpha = 0$

$P_h = P_a = 6.176$ kip/ft

$P_v = 0$

Part a: Factor of Safety Against Overturning

The following table can now be prepared to obtain ΣM_R:

Area (from figure 7. 12)	Weight (kip)	Moment arm from C(ft)	Moment about C (kip/ft)
1	$\frac{1}{2}(0.8)(15)(\gamma_c) = 0.9$	$1.25 + \frac{2}{3}(0.8) = 1.783$	1.605
2	$(1.5)(15)(\gamma_c) = 3.375$	$1.25 + 0.8 + 0.75 = 2.8$	9.45
3	$\frac{1}{2}(5.25)(15)(\gamma_c) = 5.906$	$1.25 + 0.8 + 1.5 + \frac{5.25}{3} = 5.3$	31.30
4	$(10.3)(2.5)(\gamma_c) = 3.863$	$\frac{10.3}{2} = 5.15$	19.89
5	$\frac{1}{2}(5.25)(15)(0.121) = 4.764$	$1.25 + 0.8 + 1.5 + \frac{2}{3}(5.25) = 7.05$	33.59
6	$(1.5)(15)(0.121) = \underline{2.723}$ 21.531	$1.25 + 0.8 + 1.5 + 5.25 + 0.75 = 9.55$	<u>26.0</u> $121.84 = M_R$

The overturning moment

$M_O = \frac{H'}{3}P_a = \left(\frac{17.5}{3}\right)(6.176) = 36.03$ kip/ft

$FS_{(\text{overturning})} = \frac{121.84}{36.03} = 3.38$

Part b: Factor of Safety Against Sliding

From equation (11), with $k_1 = k_2 = \frac{2}{3}$ and assuming that $P_p = 0$,

$FS_{(\text{sliding})} = \frac{\Sigma V \tan\left(\frac{2}{3}\right)\phi_2 + B\left(\frac{2}{3}\right)c_2}{P_a}$

$= \frac{21.531 \tan\left(\frac{2\times 20}{3}\right) + 10.3\left(\frac{2}{3}\right)(1.0)}{6.176}$

$= \frac{5.1 + 6.87}{6.176} = 1.94$

Part c: Pressure on the Soil at the Toe and Heel

From equations (16, 17 and 18),

$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_O}{\Sigma V} = \frac{10.3}{2} - \frac{121.84 - 36.03}{21.531} = 5.15 - 3.99 = 1.16$ ft

$q_{\text{toe}} = \frac{\Sigma V}{B}\left[1 + \frac{6e}{B}\right] = \frac{21.531}{10.3}\left[1 + \frac{(6)(1.16)}{10.3}\right] = 3.5$ kip/ft^2

$q_{\text{heel}} = \frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right] = \frac{21.531}{10.3}\left[1 - \frac{(6)(1.16)}{10.3}\right] = 0.678$ kip/ft^2

❑ COMMENTS RELATING TO STABILITY

When a weak soil layer is located at a shallow depth-that is, within a depth of about 1.5 times the width of the retaining wall-the bearing capacity of the weak layer should be carefully investigated. The possibility of excessive settlement also should be considered. In some cases, the use of lightweight backfill material behind the retaining wall may solve the problem.

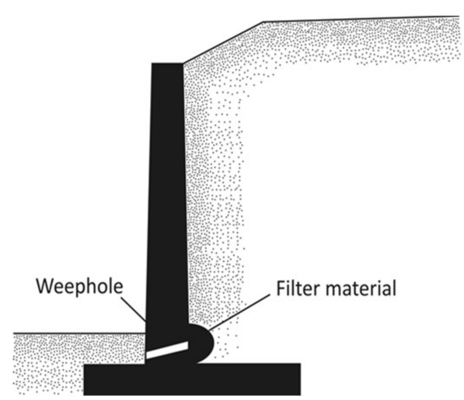
In many instances, piles are used to transmit the foundation load to a firmer layer. However, often the thrust of the sliding wedge of soil, in the case of deep shear failure, bends the piles and eventually causes them to fail. Careful attention should be given to this possibility when considering the option of pile foundations for retaining walls. (Pile foundations may be required for bridge abutments to avoid the problem of scouring).

The active state of the backfill can be established only if the wall yields sufficiently, which happen in all cases. The degree of wall yielding will depend on its height and the section modulus. Furthermore, the lateral force of the backfill will depend on several factors, as identified by Casagrande (1973):

a. Effect of temperature,
b. Groundwater fluctuation
c. Readjustment of the soil particles due to creep and prolonged rainfall
d. Tidal changes
e. Heavy wave action
f. Traffic vibration
g. Earthquakes

❑ DRAINAGE FROM THE BACKFILL OF THE RETAINING WALL

As the result of rainfall or other wet conditions, the backfill material for a retaining wall may become saturated. Saturation will increase the pressure on the wall and may create an unstable condition. For this reason, adequate drainage must be provided by means of *weepholes* and/or *perforated drainage pipes* The *weepholes*, if provided, should have a minimum diameter of about 4 in. (0.1 m) and be adequately spaced. Note that there is always a possibility that the backfill material may be washed into weepholes or drainage pipes and ultimately clog them. Thus a filter material needs to be placed behind the weepholes or around the drainage piles, as the case may be; geotextiles now were that purpose. Whenever granular soil is used as a filter, the principles. Should be followed.

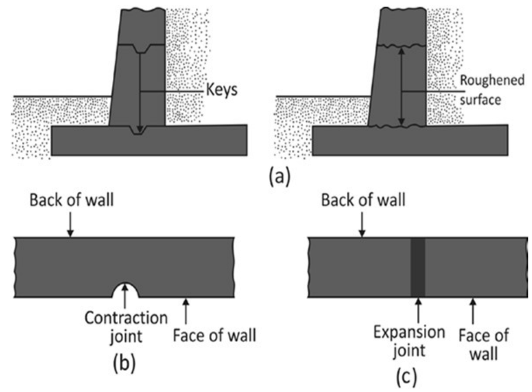


Drainage provisions for the backfill of a retaining wall

PROVISION OF JOINTS IN RETAINING-WALL CONSTRUCTION

A retaining wall may be constructed with one or more the following joints:

1. *Construction joints :* are vertical and horizontal joints that are placed between two successive pours of concrete. To increase the shear at the joints, keys may be used. If keys are not used, the surface of the first pour is cleaned and roughened before the next pour of concrete.
2. *Contraction joints:* are vertical joints (grooves) placed in the face of a wall (from the top of the base slab to the top of the wall) allow the concrete to shrink without noticeable harm. The grooves may be about 0.25 to 0.3 (≈ 6 to 8 mm) wide and 0.5 to 0.6 in. (≈ 12 to 16 mm) deep.

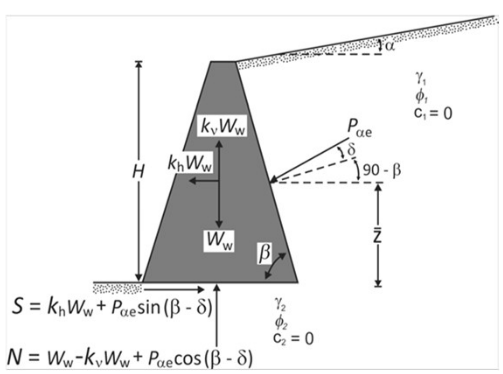


(a) Construction joints; (b) contraction joint; (c) expansion joint

3. Expansion joints : allow for the expansion of concrete caused by temperature changes; vertical expansion joints from the base to the top of the wall may also be used. These joints may be filled with flexible joint filers. In most cases, horizontal reinforcing steel bars running across the stem are continuous through all joints. The steel is greased to allow the concrete to expand.

❑ GRAVITY RETAINING-WALL DESIGN FOR EARTHQUAKE CONDITIONS

Even in mild earthquakes, most retaining walls undergo limited lateral displacement. Richards and Elms (1979) proposed a procedure for designing gravity retaining walls for earthquake conditions that allows limited lateral displacement. This procedure takes into consideration the wall inertia effect. a retaining wall with various forces acting on it, which are as follows (per unit length of the wall):



Stability of a retaining wall under earthquake forces

a. W_w = weight of the wall
b. P_a = active force with earthquake condition taken into consideration

The backfill of the wall and the soil on which the wall is retaining are assumed cohesionless. Considering the equilibrium of the wall, it can be shown that

$$W_w = \left[\tfrac{1}{2}\gamma_1 H^2 (1 - k_v) K_{ae}\right] C_{IE}$$

Where γ_1 = unit weight of the backfill

$$C_{IE} = \frac{\sin(\beta - \delta)\cos(\beta - \delta)\tan\phi_2}{(1 - k_v)(\tan\phi_2 - \tan\theta')}$$

And

$$\theta' = \tan^{-1}\left(\frac{k_h}{(1 - k_v)}\right)$$

Based on the above equations, the following procedure may be used to determine the weight of the retaining wall, W_w, for tolerable displacement that may take place during an earthquake.

1. Determine the tolerable displacement of the wall, Δ.
2. Obtain a design value of k_h from

$$k_h = A_a \left(\frac{0.24_v^2}{A_a \Delta}\right)^{0.25}$$

In above equation A_a and A_v are effective acceleration coefficients and Δ is displacement in inches. The magnitudes of A_a and A_v are given by the Applied Technology Council (1978) for various regions of the United States.

3. Assume that $k_v = 0$, and, with the value of k_h obtained, calculate K_a
4. Use the value of K_a determined in step 3 to obtain the weight of the wall (W_w).
5. Apply a factor of safety to the value of W_w obtained in step 4.

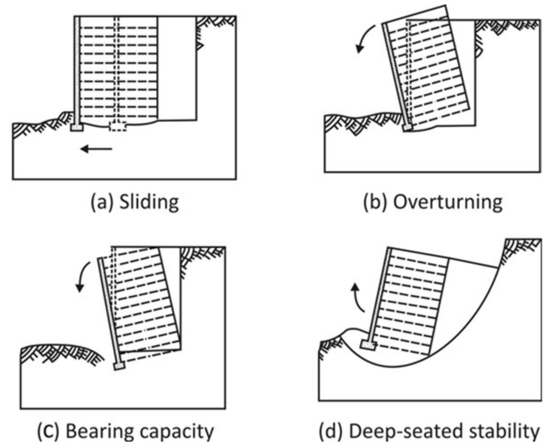
❑ MECHANICALLY STABILIZED RETAINING WALLS

GENERAL DESIGN CONSIDERATIONS

The general design procedure of any mechanically stabilized retaining wall can be divided into two parts:

1. Satisfying *internal stability* requirements
2. Checking the *external stability* of the wall

The internal stability checks involve determining tension and pullout resistance in the reinforcing elements and the integrity of facing elements. The external stability checks include checks for overturning, sliding and bearing capacity failure. The following sections will discuss the retaining wall design procedures with metallic strips, geotextiles, and geogrids.



External stability checks (after Transportation Research Board 1995)

RETAINING WALLS WITH METALLIC STRIP REINFORCEMENT

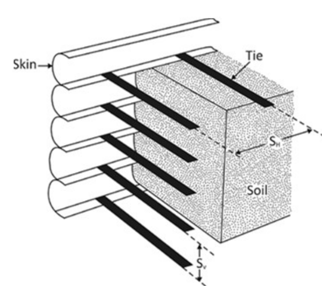
Reinforced earth walls are flexible walls. Their main components are

1. Backfill, which is granular soil
2. Reinforcing strips, which are thin, wide strips placed at regular intervals
3. A cover on the front face, which is referred to as the *skin*

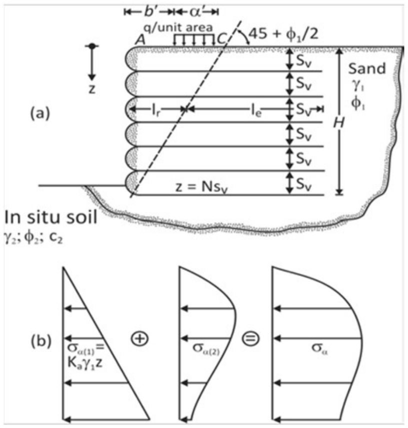
at any depth, the reinforcing stripes or ties are placed with a horizontal spacing of SS_{HH} center-to-center; the vertical spacing of the strips or ties is SS_{VV} center-to-center. The skin can be constructed with sections of relatively flexible thin material. Lee et al. (1973) showed that, with a conservative design, a 0.2-in.thick (≈ 5 mm) galvanized steel skin would be enough to hold a wall about 45-50 ft (14-15 m) high. In most cases, precast concrete slabs can be used as skin. The slabs are grooved to fit into each other so that soil cannot flow out between the joints. When metal skins are used, they are bolted together, and reinforcing strips are placed between the skins.

Calculation of Active Horizontal and vertical Pressure

a retaining wall with a granular backfill having a unit weight of γ_1 and a friction angle of ϕ_1. Below the base of the retaining wall, the *in situ* soil has been excavated and reccopacted, with granular soil used as backfill. Below the backfill, the *in situ* soil has a unit weight of γ_2 and a friction angle of ϕ_2, and cohesion of c_2. A surcharge having an intensity of q per unit area lies atop the retaining wall. The wall has reinforcement ties at depths $z = 0, S_V, \ldots, NS_V$. The height of the wall is $NS_V = H$.



Reinforced earth retaining wall



Analysis of a reinforced earth retaining wall

According to the Rankine active pressure theory

$\sigma_a = \sigma_v K_a - 2cK_a$

Where

σ_a = Rankine active pressure at any depth zz

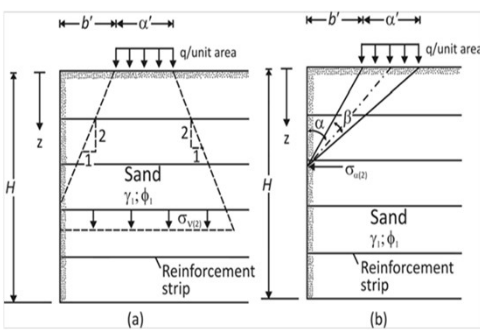
For dry granular soils with no surcharge at the top, $cc = 0$, $\sigma_v = \gamma_1 z$, and $K_a = \tan^2(45 - \phi_1/2)$. Thus

$$\sigma_{a(1)} = \gamma_1 z K_a$$

When a surcharge is added at the top, as shown in figure 1.26

$$\sigma_v = \underbrace{\sigma_{v(1)}}_{\text{Due to soil only}} + \underbrace{\sigma_{v(2)}}_{\text{Due to the surcharge}}$$

$= \gamma_1 z$



(a) notation for the relationship of $\sigma_{v(2)}$; (b) notation for the relationship of $\sigma_{a(2)}$

$$\sigma_{v(2)} = \frac{qa'}{a' + z} \quad (\text{for} z \le 2b')$$

And

$$\sigma_{v(2)} = \frac{qa'}{a' + \frac{z}{2} + b'} \quad (\text{for} z > 2b')$$

Also, when a surcharge is added at the top, the lateral pressure at any depth is

$$\sigma_a = \underbrace{\sigma_{a(1)}}_{\text{Due to soil only}} + \underbrace{\sigma_{a(2)}}_{\text{Due to the surcharge}}$$

$= K_a \gamma_1 z$

According to Laba and Kennedy (1986), $\sigma_{a(2)}$ may be expressed

$$\sigma_{a(2)} = M\left[\frac{2q}{\pi}(\underbrace{\beta}_{\text{(in radius)}} - \sin\beta\cos 2\alpha)\right]$$

Where

$$M = 1.4 - \frac{0.4b'}{0.14H} \ge 1$$

Tie Force

The tie force *per unit length of the wall* developed at any depth z is

T = active earth pressure at depth zz × area of the wall to be supported by the tie

= $(\sigma_a)\ (S_V S_H)$

Factor of Safety Against Tie Failure

The reinforcement ties at each level and thus the walls cold fail by either (a) tie breaking or (b) tie pullout.

The factor of safety against *tie breaking* may be determined as

$FS_{(B)}$ = $\frac{\text{yield or breaking strength of each tie maximum tie force in any tie}}{\text{Maximum tie force in any tie}}$

= $\frac{wt\,fy}{\sigma_a S_V S_H}$

Where

w = width of each tie

t = thickness of each tie

f_y = yield or breaking strength of the tie material

A factor of safety of about 2.5-3 is generally recommended for ties at all levels.

Reinforcing ties at any depth, z, will fail by pullout if the frictional resistance developed along their surfaces is less than the force to which the ties are being subjected. The *effective length* of the ties along which the frictional resistance is developed may be conservatively taken as the length that extends *beyond the limits of the Rankine active failure zone,* which is the zone ABA in figure 29. Line BA in figure 29 makes an angle of $45 + \phi_1/2$ with the horizontal. Now, the maximum friction force F_R that can be realized for a tie at depth z is

$F_R = 2l_e w\sigma_v \tan\phi_\mu$

Where

l_e = effective length

σ_v = effective vertical pressure at a depth z

ϕ_μ = soil − tie friction angle

Thus the factor of safety against *tie pullout* at any depth z is

$FS\ (P) = FRT$

Where $FS(P)$ = factor of safety against tie pull-out

hence, $FS(P) = \frac{2\ le\ w\ \sigma v \tan \phi\mu}{\sigma aSVSH}$

Total Length of Tie

The total length of ties at any depth is

$L = l_r + l_e$

Where

l_r = length with the Rankine failure zone

l_e = effective length

For a given $FS_{(P)}$

$$l_e = \frac{FS_{(P)}\sigma_a S_V S_H}{2w\sigma_v \tan\phi_\mu}$$

Again, at any depth z,

$$l_r = \frac{(H-z)}{\tan\left(45+\frac{\phi_1}{2}\right)}$$

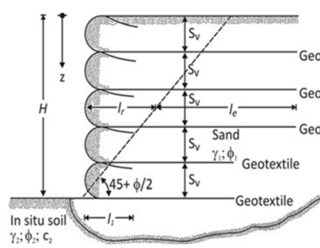
$$L = \frac{(H-z)}{\tan\left(45+\frac{\phi_1}{2}\right)} + \frac{FS_{(P)}\sigma_a S_V S_H}{2w\sigma_v \tan\phi_\mu}$$

1. RETAINING WALLS WITH METALLIC STRIP REINFORCEMENT
2. Calculation of Active Horizontal and vertical Pressure
   - Tie Force
   - Factor of Safety Against Tie Failure
   - Total Length of Tie
3. STEP-BY-STEP DESIGN PROCEDURE (METALLIC STRIP REINFORCEMENT
   - General:
   - Internal Stability:
   - Internal Stability Check
   - Check for overturning:
   - Check for sliding
   - Check for bearing capacity

RETAINING WALLS WITH GEOTEXTILE REINFORCEMENT

In this type of retaining wall, the facing of the wall is formed by lapping the sheets as shown with a lap length of l_1. When construction of the wall is finished, the exposed face of the wall must be covered; otherwise, the geotextile will deteriorate from exposure to ultraviolet light. *Bitumen emulsion* or *Gunite* is sprayed on the wall face. A wire mesh anchored to the geotextile facing may be necessary to keep the coating on the face of the wall.

Following is a step-by-step procedure for design based on the recommendations of Bell et al. (1975) and Koerner (1990).



Retaining wall with geotextile reinforcement

Internal Stability:

1. Determine the active pressure distribution on the wall from

   $\sigma_a = K_a\sigma_v = K_a\gamma_1 z$

   Where

   K = Rankine earth pressure coefficient = $\tan^2(45 - \phi_1/2)$
   γ_1 = unit weight of the granular backfill
   ϕ_1 = friction angle of the granular backfill

2. Select a geotextile fabric that has an allowable strength of σ_G (lb/ft or kN/m).
3. Determine the vertical spacing of the layers at any depth z from

$$S_V = \frac{\sigma_G}{\sigma_a FS_{(B)}} = \frac{\sigma_G}{(\gamma_1 z K_a)[FS_{(B)}]}$$

The magnitude of $FS_{(B)}$ is generally 1.3-1.5.

4. Determine the length of each layer of geotextile from

$$L = l_r + l_e$$

Where $l_r = \frac{H-z}{\tan\left(45+\frac{\phi_1}{2}\right)}$

$l_e = \frac{S_V \sigma_a [FS_{(P)}]}{2\sigma_v \tan \phi_F}$

$\sigma_a = \gamma_1 z K_a$

$\sigma_v = \gamma_1 z$

$FS_{(P)} = 1.3$ to 1.5

ϕ_F = friction angle at geotextile – soil interface

≈ 2/3 ϕ_1

✓ Based on the published results, the assumption of ϕ_F / ϕ_1 ≈ 2/3 is reasonable and appears to be conservative.

Martin et al. (1984) presented the following laboratory test results for ϕ_F/ ϕ_1 between various types of geotextiles and sand.

Type	ϕ_F/ ϕ_1
Woven-monofilament/concrete sand	0.87
Woven-silt film/concrete sand	0.8
Woven-silt film/rounded sand	0.86
Woven-silt film/silty sand	0.92
Nonwoven-melt-bonded/concrete sand	0.87
Nonwoven-needle/concrete sand	1.0
Nonwoven-needle-punched/rounded sand	0.93
Nonwoven-needle-punched/silty sand	0.91

5. Determine the lap length, l_1 from

$$l_1 = \frac{S_V \sigma_a FS_{(P)}}{4\sigma_v \tan \phi_F}$$

The minimum lap length should be 3 ft (1 m).

External Stability:

6. Check the factors of safety against overturning, sliding, and bearing capacity failure.

REFERENCE

NPTEL: ADVANCED FOUNDATION ENGINEERING
LECTURE 24-28

UNIT-II
MCE-164
Geotechnics of Hill Area

Course Instructor: Dr. V. B. Chauhan

SLOPE STABILITY ANALYSIS

The stability of rock slopes is greatly controlled by the shear strength along the joints and interfaces between the unstable rock block/wedge and intact rock, as well as by the geometric interaction of jointing and bedding patterns in the rock mass constituting the slope. The magnitude of the available shear strength along joints and interfaces is very difficult to determine due to the inherent variability of the material and the difficulties associated with sampling and laboratory testing.

<u>Factors that directly or indirectly influence the strength include the following:</u>

1. The planarity and smoothness of the joint's surfaces. A smooth planar surface will have a lower strength than an irregular and rough surface.

2. The inclination of the discontinuity plane with respect to the slope.

3. The openness of the discontinuity, which can range from a small fissure to a readily visible joint.

4. The extent of the weathering along the surfaces and the possible infill of the joint with weaker material such as clays and calcareous materials. A calcareous infill may potentially increase the strength of the joint, whereas a soft clay infill may reduce the strength of the joint to the same level as the clay material itself. Such infills may also change the seepage pattern, improving or degrading the drainage, which will be manifested by an increase or decrease in pore water pressures within the joints.

Factor of safety

$$FS = \frac{F_r}{F_i}$$

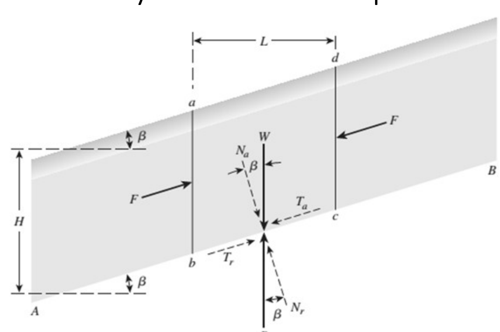
where Fr is the total force available to resist the sliding of the rock block and Fi is the total force tending to induce sliding.

For a slope on the point of failure, a condition of limiting equilibrium exists in which Fr = Fi, and thus FS =1.
For stable slopes, Fr> Fi, and therefore FS >1.

In practice, rock slopes with FS =1.3 to 1.5 are considered to be stable; the lower value is taken for temporary slopes such as mine slopes, whereas the higher value is considered for permanent slopes such as slopes adjacent to road pavements and railway tracks.

3

Stability of Infinite Slopes



Slopes can be considered as infinite in the case of large land slides, where the solid mass is Moving approximately parallel to the ground surface or the face of the slope.

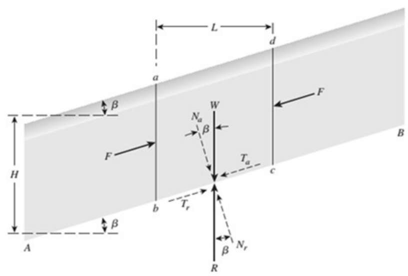
A planar slip surface is assumed. The slope extends infinitely in the lateral and longitudinal directions, and The length of the slide is very long relative to the depth or height of the sliding surface.

$$W = (\text{Volume of soil element}) \times (\text{Unit weight of soil}) = \gamma LH$$

1. Force perpendicular to the plane $AB = N_a = W \cos \beta = \gamma LH \cos \beta$.
2. Force parallel to the plane $AB = T_a = W \sin \beta = \gamma LH \sin \beta$. Note that this is the force that tends to cause the slip along the plane.

4

Stability of Infinite Slopes



$$\sigma' = \frac{N_a}{\text{Area of base}} = \frac{\gamma LH \cos\beta}{\left(\dfrac{L}{\cos\beta}\right)} = \gamma H \cos^2\beta$$

$$\tau = \frac{T_a}{\text{Area of base}} = \frac{\gamma LH \sin\beta}{\left(\dfrac{L}{\cos\beta}\right)} = \gamma H \cos\beta \sin\beta$$

$$N_r = R\cos\beta = W\cos\beta$$

$$T_r = R\sin\beta = W\sin\beta$$

The resistive shear stress that develops at the base of the element is equal to

$$(T_r)/(\text{Area of base}) = \gamma H \sin\beta\cos\beta.$$

$$\tau_d = c'_d + \gamma H \cos^2\beta \tan\phi'_d$$

$$\tau_d = c'_d + \sigma' \tan\phi'_d$$

Stability of Infinite Slopes

If a soil possesses cohesion and friction, the depth of the plane along which critical equilibrium occurs may be determined by substituting Fs =1 and H=Hcr

$$\gamma H \sin\beta\cos\beta = c'_d + \gamma H\cos^2\beta\tan\phi'_d$$

$$\frac{c'_d}{\gamma H} = \sin\beta\cos\beta - \cos^2\beta\tan\phi'_d$$

$$= \cos^2\beta(\tan\beta - \tan\phi'_d)$$

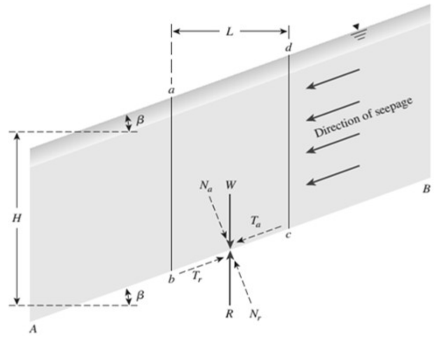
$$\tan\phi'_d = \frac{\tan\phi'}{F_s} \quad \text{and} \quad c'_d = \frac{c'}{F_s}$$

$$F_s = \frac{c'}{\gamma H\cos^2\beta\tan\beta} + \frac{\tan\phi'}{\tan\beta}$$

$$H_{cr} = \frac{c'}{\gamma}\frac{1}{\cos^2\beta(\tan\beta - \tan\phi')}$$

For granular soils, $c' = 0$, and the factor of safety, F_s, becomes equal to $(\tan\phi')/(\tan\beta)$. This indicates that in an infinite slope in sand, the value of F_s is independent of the height H and the slope is stable as long as $\beta < \phi'$.

Stability of Infinite Slopes with steady state seepage



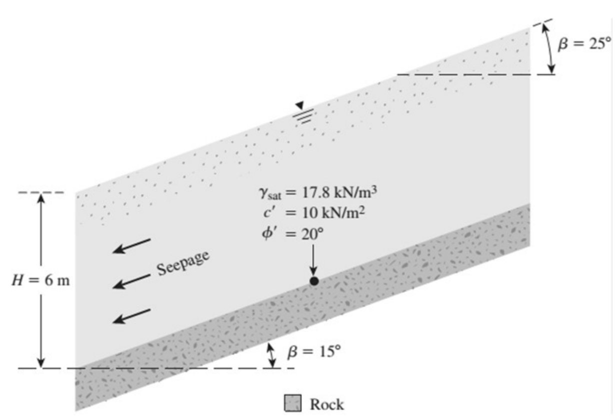
If there is steady state seepage through the soil and the ground water table coincides with the ground surface

$$F_s = \frac{c'}{\gamma_{sat} H \cos^2 \beta \tan \beta} + \frac{\gamma' \tan \phi'}{\gamma_{sat} \tan \beta}$$

Numerical Problem

For the infinite slope with a steady state seepage, determine:

a. The factor of safety against sliding along the soil-rock interface.
b. The height, H, that will give a factor of safety (F_s) of 2 against sliding along the soil-rock interface.



SOLUTION

$$F_s = \frac{c'}{\gamma_{sat} H \cos^2\beta \tan\beta} + \frac{\gamma' \tan\phi'}{\gamma_{sat}\tan\beta}$$

$$\gamma_{sat} = 17.8 \text{ kN/m}^3$$

$$\gamma' = \gamma_{sat} - \gamma_w = 17.8 - 9.81 = 7.99 \text{ kN/m}^3$$

$$F_s = \frac{10}{(17.8)(6)(\cos 15)^2(\tan 15)} + \frac{7.99 \tan 20}{17.8 \tan 15} = 0.375 + 0.61 = \mathbf{0.985}$$

$$F_s = \frac{c'}{\gamma_{sat} H \cos^2\beta \tan\beta} + \frac{\gamma' \tan\phi'}{\gamma_{sat}\tan\beta}$$

$$2 = \frac{10}{(17.8)(H)(\cos 15)^2(\tan 15)} + \frac{7.99 \tan 20}{17.8 \tan 15} = \frac{2.247}{H} + 0.61$$

$$H = \frac{2.247}{2 - 0.61} = \mathbf{1.62\ m}$$

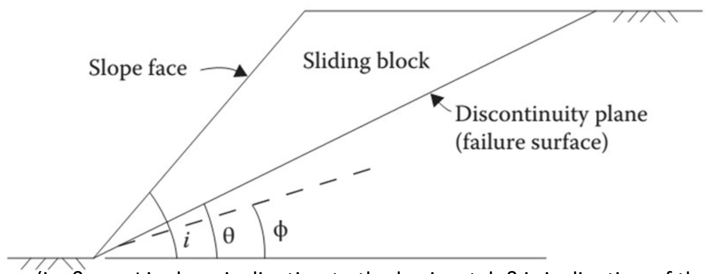
Rock slope stability

- Rock slopes either occur naturally or are engineered by people as products of excavations to create space for buildings, highways and railway tracks, powerhouses, dams and mine pits.
- Objective: Basic modes/mechanisms of rock slope failures and and the fundamental concepts and methods of rock slope stability analysis
- MODES OF ROCK SLOPE FAILURE:
- The modes of rock slope failure depend mainly on the geometric interaction of existing discontinuities (jointing and bedding patterns) and free space/excavation surfaces in the rock mass constituting the slope.
- For safe and economic design of rock slopes, it is important to recognize the modes/mechanisms in which slopes in rock masses can fail.
- The spherical presentation of geological data (dip and strike) helps identify the most likely basic potential modes of rock slope failure

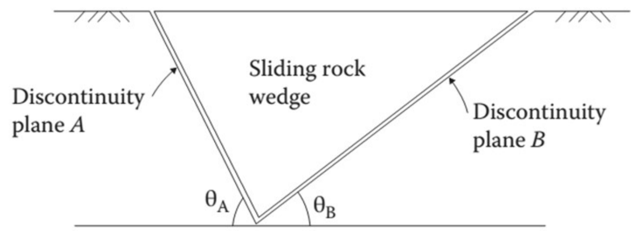
engineered (excavated) rock slope

natural rock slope

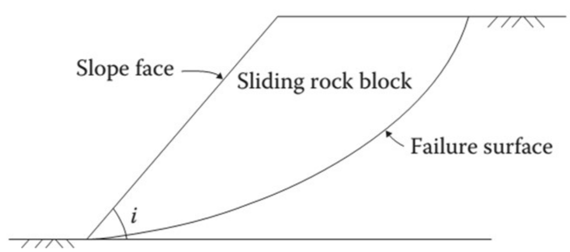
- 1. Plane failure: In plane failure mode, the rock block slides on a single face that can be a joint plane or bedding plane striking parallel to the slope face and dipping into free space/excavation at an angle greater than the angle of internal friction of the joint/bedding material
- 2. Wedge failure: the wedge of rock slides simultaneously on two discontinuity planes, striking obliquely across the slope face, along their line of intersection daylighting into the slope face, provided that the inclination of this line is significantly greater than the average angle of internal friction of the two joint/ bedding materials
- 3. Circular failure: the heavily jointed and weathered rock mass, similar to a waste dump rock, slides on a single cylindrical face into free space/excavation
- 4. Toppling failure: the multiple rock columns/layers caused by a steeply dipping joint set rotate about their bases into the free space/excavation.
- Plane and wedge failures are more common than circular and toppling failures. Toppling failure can be very significant, if not dominant, in some rock types of steep mountain slopes or open pit mines



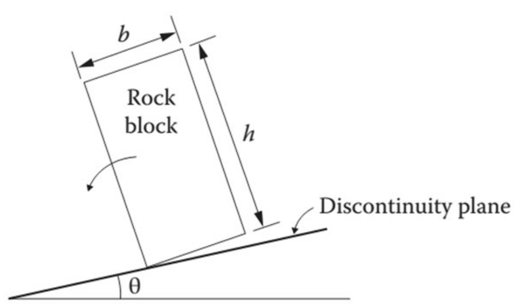
Plane failure (i > θ > φ; I is slope inclination to the horizontal, θ is inclination of the discontinuity plane/failure plane and φ is angle of internal friction of the joint/bedding material)



Wedge failure (θ_A and θ_B are the inclinations of discontinuity planes A and B, respectively);



(c) Circular failure



(d) Toppling failure (b is width and h is height of the rock block)

Basic modes of rock slope failure

Mode of rock failure	*Description*	*Typical materials*
Plane failure	Sliding without rotation along a face; single or multiple blocks	Hard or soft rocks with well-defined discontinuities and jointing, e.g. layered sedimentary rocks, volcanic flow rocks, block-jointed granite, foliated metamorphic rocks
Wedge failure	Sliding without rotation on two nonparallel planes, parallel to their line of intersection; single or multiple blocks	Blocky rocks with at least two continuous and nonparallel joint sets, e.g. cross-jointed sedimentary rocks, regularly faulted rocks, block-jointed granite and especially foliated or jointed metamorphic rocks
Circular failure	Sliding on a cylindrical face	Heavily jointed and weathered rock masses similar to the soils
Toppling failure	Forward rotation about an edge/base; single or multiple blocks	Hard rocks with regular, parallel joints dipping away from the free space/ excavation, i.e. dipping into the hillside, with or without crossing joints; foliated metamorphic rocks and steeply dipping layered sedimentary rocks; also in block-jointed granites

SLOPE STABILITY ANALYSIS

The stability of rock slopes is greatly controlled by the shear strength along the joints and interfaces between the unstable rock block/wedge and intact rock, as well as by the geometric interaction of jointing and bedding patterns in the rock mass constituting the slope. The magnitude of the available shear strength along joints and interfaces is very difficult to determine due to the inherent variability of the material and the difficulties associated with sampling and laboratory testing.

Factors that directly or indirectly influence the strength include the following:

1. The planarity and smoothness of the joint's surfaces. A smooth planar surface will have a lower strength than an irregular and rough surface.

2. The inclination of the discontinuity plane with respect to the slope.

3. The openness of the discontinuity, which can range from a small fissure to a readily visible joint.

4. The extent of the weathering along the surfaces and the possible infill of the joint with weaker material such as clays and calcareous materials. A calcareous infill may potentially increase the strength of the joint, whereas a soft clay infill may reduce the strength of the joint to the same level as the clay material itself. Such infills may also change the seepage pattern, improving or degrading the drainage, which will be manifested by an increase or decrease in pore water pressures within the joints.

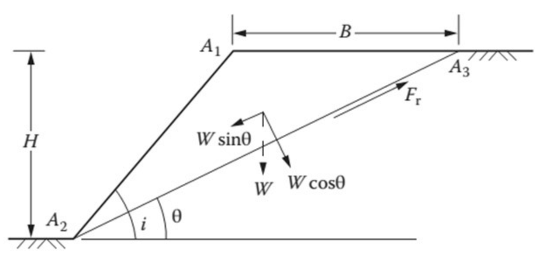
Factor of safety $FS = \frac{F_r}{F_i}$

where Fr is the total force available to resist the sliding of the rock block and Fi is the total force tending to induce sliding.

For a slope on the point of failure, a condition of limiting equilibrium exists in which Fr = Fi, and thus FS =1.
For stable slopes, Fr> Fi, and therefore FS >1.

In practice, rock slopes with FS =1.3 to 1.5 are considered to be stable; the lower value is taken for temporary slopes such as mine slopes, whereas the higher value is considered for permanent slopes such as slopes adjacent to road pavements and railway tracks.

Plane failure



Analyzed as a two-dimensional limit equilibrium problem, considering a slice of unit thickness through the slope. Only the force equilibrium is considered, neglecting any resistance to sliding at the lateral boundaries of the sliding block. The joint/bedding plane material is assumed to be a c–φ soil material, with c and φ as cohesion and angle of internal friction (also called angle of shearing resistance), respectively, obeying the Mohr–Coulomb failure criterion.

The total force available to resist the sliding block is

$$F_r = sA$$

where s is the shear strength of the sliding failure plane, and A is the area of the base A2A3 of the sliding rock block.

$$A = \frac{H}{\sin\theta}$$

The top width B is calculated

$$B = H(\cot\theta - \cot i) = \frac{H\sin(i-\theta)}{\sin i \sin\theta}$$

The Mohr–Coulomb failure cri

$$s = c + \sigma_n \tan\phi$$

Where σ_n is the normal stress on the failure plane

$$F_r = cA + F_n \tan\phi$$

$$F_n = \sigma_n A$$

Considering equilibrium of forces acting on the rock block in a direction normal to the slope face, F_n is obtained as

$$F_n = W\cos\theta$$

The weight W is calculated as

$$W = \frac{1}{2}\gamma BH$$

$$W = \frac{1}{2}\left[\frac{\sin(i-\theta)}{\sin i \sin\theta}\right]\gamma H^2$$

$$F_r = \frac{cH}{\sin\theta} + \frac{1}{2}\left[\frac{\sin(i-\theta)\cos\theta}{\sin i \sin\theta}\right]\gamma H^2 \tan\phi$$

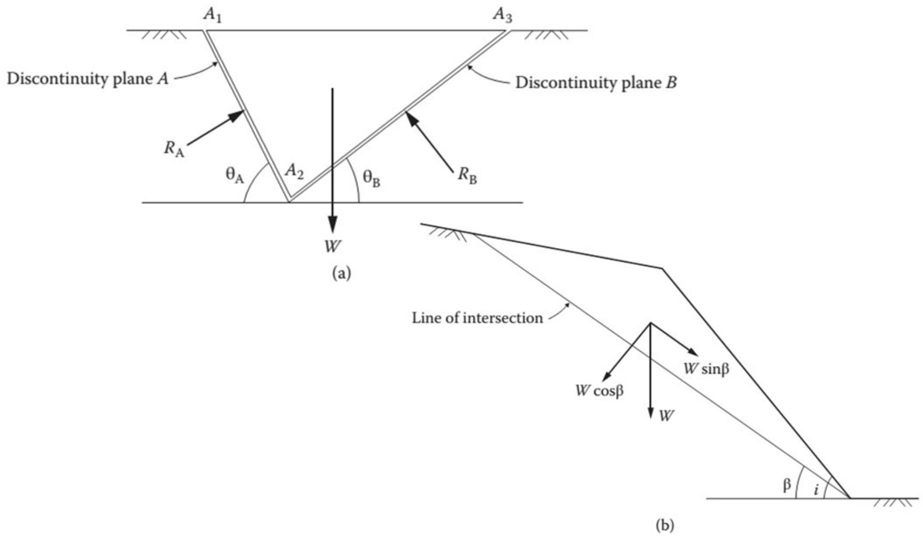
$$F_i = W\sin\theta$$

$$F_i = \frac{1}{2}\left[\frac{\sin(i-\theta)}{\sin i}\right]\gamma H^2$$

$$\mathrm{FS} = \frac{2c\sin i}{\gamma H \sin\theta \sin(i-\theta)} + \frac{\tan\phi}{\tan\theta}$$

$$\mathrm{FS} = \frac{2c^* \sin i}{\sin\theta\sin(i-\theta)} + \frac{\tan\phi}{\tan\theta} \qquad c^* = c/\gamma H$$

WEDGE FAILURE



Forces acting on the rock wedge: (a) view of wedge looking at its face showing definition of angles θ_A and θ_B, and reactions R_A and R_B of discontinuity planes A and B, respectively; (b) cross section of wedge showing resolution of the weight W.

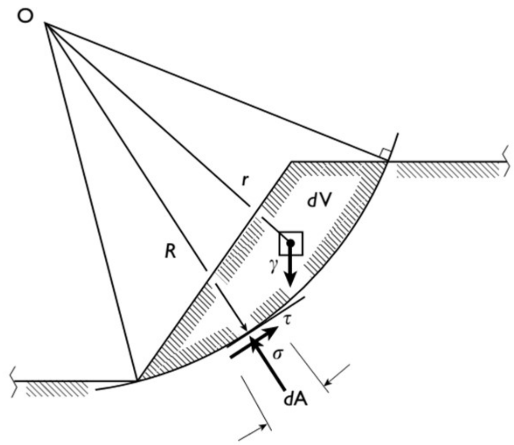
ROTATIONAL SLOPE FAILURES

(1) A conventional reverse rotation along a surface that is often approximated as a circular arc transecting a soil-like material

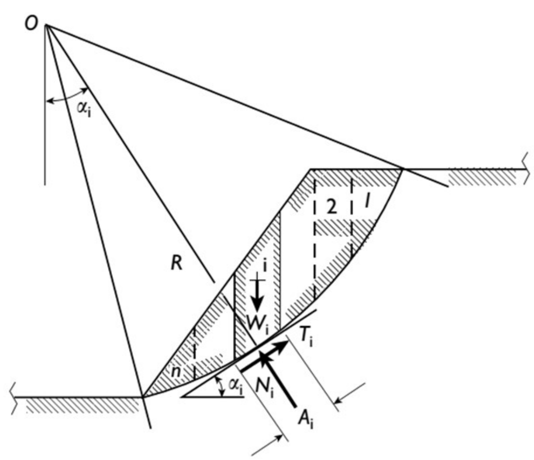
(2) A forward rotation associated with toppling of rock blocks.

$$FS = \frac{M_R}{M_D}$$

Moments from surface and body forces for a slope failure along a circular arc



Circular arc slope failure and subdivision into slices. Slices are numbered 1, 2, …,n



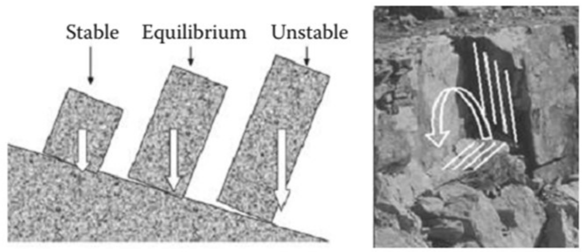
$$\sum_{1}^{n} R\sin(\alpha_i)W_i = \sum_{1}^{n} RT_i(\text{stress}) = \sum_{1}^{n} RT_i(\text{strength})/\text{fs}_i$$

$$\text{FS} = \frac{\sum_1^n RT_i(\text{strength})}{\sum_1^n R\sin(\alpha_i)W_i} = \frac{\sum_1^n N_i' \tan(\phi_i) + c_i A_i}{\sum_1^n \sin(\alpha_i)W_i}$$

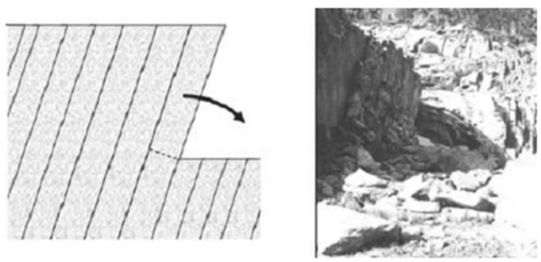
$$\text{FS} = \frac{\sum_i [(W_n - P)\tan(\phi) + cA]}{\sum_i W_s}$$

Toppling failure

- Toppling failures occur in a wide range of rock masses in both natural and engineered slopes. They involve the rotation of columns or blocks of rocks about their bases.
- The simplest toppling mechanisms involve a single block, resulting in single-block toppling or flexural toppling.
- The single-block toppling occurs when the rock block is already detached from the rock mass of the slope.
- The flexural toppling occurs when the rock block remains attached to the rock mass of the slope.

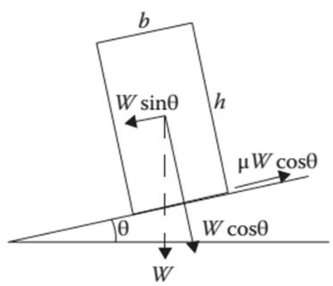


(a)



(b)

Simple toppling failures involving a single block: (a) single-block toppling; (b) single-block flexural toppling.



For a single-rock block resting on a discontinuity plane, if the width b and height h of the rock block are such that its weight acts outside its base, then there is a potential for the block to topple.

For this condition to occur, the resisting moment about the outer lower edge of the block should be less than the driving moment about the same edge; that is,

$$(W\cos\theta)\left(\frac{b}{2}\right) < (W\sin\theta)\left(\frac{b}{2}\right)$$

OR

$$\frac{b}{h} < \tan\theta$$

For the sliding of the block

$$W \sin\theta > \mu W \cos\theta$$

$$\tan\theta > \mu$$

where μ is the coefficient of friction between the sliding block and the joint/bedding plane. Since μ =tan φ,

$$\theta > \phi$$

Following four conditions for toppling and/or sliding of the block:

- Toppling only: $\frac{b}{h} < \tan\theta$ and $\theta < \phi$
- Toppling with sliding: $\frac{b}{h} < \tan\theta$ and $\theta > \phi$
- Sliding only: $\frac{b}{h} > \tan\theta$ and $\theta > \phi$
- No toppling and sliding, that is stable: $\frac{b}{h} > \tan\theta$ and $\theta < \phi$

GROUTING

GROUTING

Grouting is the process of placing a material into cavities in concrete or masonry structure for the purpose of increasing the load bearing capacity of a structure, restoring the monolithic nature of a structural member, filling voids around pre cast connections steel base plates, providing fire stops, stopping leakages, placing adhesives and soil stabilization.

GROUT is a mixture of water, cement and optional material like sand, water reducing admixtures, expansion agents and pozzolans. The water to cement ratio is around 0.5. Fine sand is used to avoid segregation.

31

Categories of grout

Suspension Grout

Liquid Grout or Solution Grout: Suspension groutis a mixture of one or several inert materials like cement, clays etc suspended in a fluid i.e water. Suspension grout is a mixture of pure cement with water.

Liquid grout or solution groutconsists of chemical products in a solution or an emulsion form and their reagents. The most frequently used products are sodium silicate and certain resins.

32

GROUTING IN SOIL

Injection of slurry or a liquid solution into a soil or rock formation is termed as grouting. The injected material is referred to as the grout. The process of grouting was developed primarily as a technique for making vertical seepage barriers beneath dams and hydraulic structures by injecting cement slurry into the void space of river bed material.

Grout is a construction material used to embed rebars in masonry walls, connect sections of pre-cast concrete, fill voids, and seal joints (like those between tiles).

Grout is generally composed of a mixture of water, cement, sand, often color tint, and sometimes fine gravel

TYPES of Grout

Suspension :- Suspensions consist of small-sized particles dispersed in a liquid medium. These include cement grouts, that is, slurry of cement in water; soilcement grouts consisting of slurry of soil and cement in water; and Bentonite grouts comprising slurry of Bentonite in water .

Emulsion :- Emulsions consist of colloidal droplets of liquid dispersed in a liquid medium; bituminous emulsion fall in this category .

Solutions:- Solutions are liquid homogenous molecular mixtures of two or more substances; chemical grouts such as sodium silicate solutions and acrylic resins are examples of solutions

Desirable Characteristics of Grout

Properties of a grout are described in terms of five parameters:-

1. Groutability:- Expresses the ability of the grout to reach the desired location in the soil, mass. To be able to do so the grout should possess sufficiently high fluidity and the suspended particles, if any, must be of a size that enables them to enter the void spaces in the soil mass.

Groutability Ratio = D_{15} of soil / $D_{85}$of grout > 25

2. Stability:- Is the capacity of the grout to remain in a fluid state and not segregate into its separate components. We need the grout to be stable until it has reached its destination. Stability of clay-cement grout is usually more than that of a cement grout.

35

Desirable Characteristics of Grout

3. Setting Time:- Is the time it takes before the grout sets into a cemented mass or gel. Early setting can cause difficulty in grout reaching its destination and late setting can result in the grout being washed away if seepage is occurring through the soil. Additives are used to retard or accelerate the setting time as required.

4. Permanence:- Indicates the resistance the grout possess against being displaced from the soil voids with time. Cement grouts have greater permanence than Bentonite grouts which can get washed away with time by seepage of water through the grouted zone.

5. Toxicity:- Is the capacity of the grout to contaminate the ground water coming in contact with it and of adversely affecting the health of workers handling and injecting the grout into the soil.

36

Applications of grouting

Repairing of cracks: The wide cracks may be repaired by filling them with portland cement grout.

The grout mixture may contain cement and water or cement, sand and water, depending upon the width of crack.

The water cement ratio should be kept as low as practicable to maximize strength and minimize shrinkage. Strengthening existing walls

The lateral strength of buildings can be improved by increasing the strength and stiffness of the existing individual walls, whether they are cracked or uncracked.

37

GROUTING METHODS

Permeation Grouting

Compaction Grouting

Jet Grouting

Soil Fracture Grouting

Circuit Grouting

Point Grouting

Electro kinetic injection

38

Permeation Grouting

• Grout fills the pores without any volume changes. Include Cement grouts, bentonite grouts and chemical grouts.

• Grouting into an open hole in self-supporting ground through pipes at the surface through an injection pipe held in place in the hole or casing

by a packer .

• From a pipe driven into the ground and withdrawn as injection proceeds

• Through a pipe left in place in the ground as with a tube

Applications:

Seepage Control

For making vertical seepage barriers beneath hydraulic structures

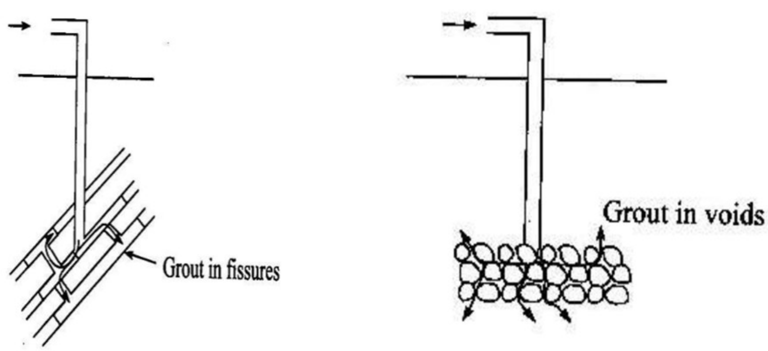
Stoppage of seepage through joints of underground structures such as tunnel lining/ basement wall, etc.

Soil Solidification and Stabilization

For stabilization of soil around tunnels and shafts

39

Permeation Grouting



(a) Permeation grouting in rocks

(b) Permeation grouting in soils

40

Compaction grouting

A good option if the foundation of an existing building requires

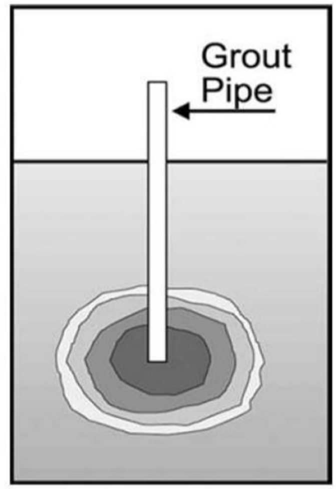
improvement, since it is possible to inject the grout from the side

or at an inclined angle to reach beneath the building

A bulb shaped grouted mass is formed.

Soil-cement grout

Can be performed as pretreatment before the structure is built



41

3. Jet Grouting

Involves the injection of low viscosity liquid grout into the pore spaces of granular soils. This creates

hardened soils to replace loose liquefiable soils

Jet grouting is used as replacement technique, in which soils ranging from silt to clay and weak rocks can be treated

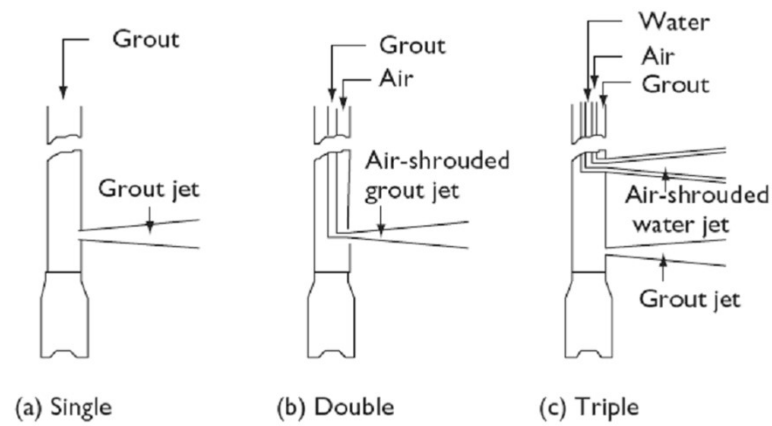
This method consists of lowering a drill pipe into a 150 mm dia bore hole

The drill pipe is specially designed which simultaneously conveys pumped water, compressed air and grout fluid.

Three systems of jet grouting Single, Double & Triple

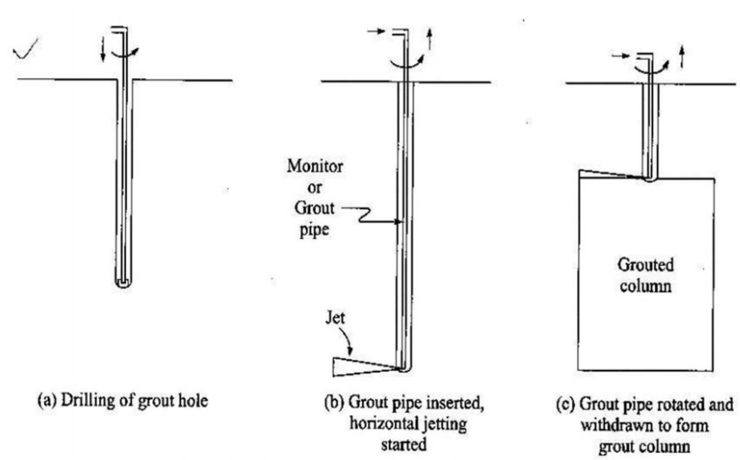
42

Systems of jet grouting



43

Sequence in Single Jet Grouting



44

Triple Jet Grouting

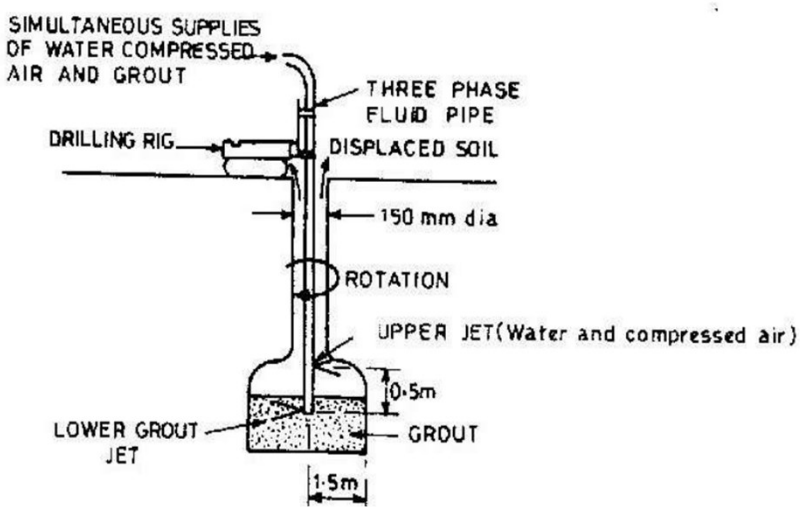
At the bottom end of the pipe two nozzles are provided at 500 mm apart.

The upper nozzle (1.8 mm diameter) delivers water surrounded by a collar of compressed air to produce a cutting jet.

The grout is delivered through the lower nozzle (7 mm dia)

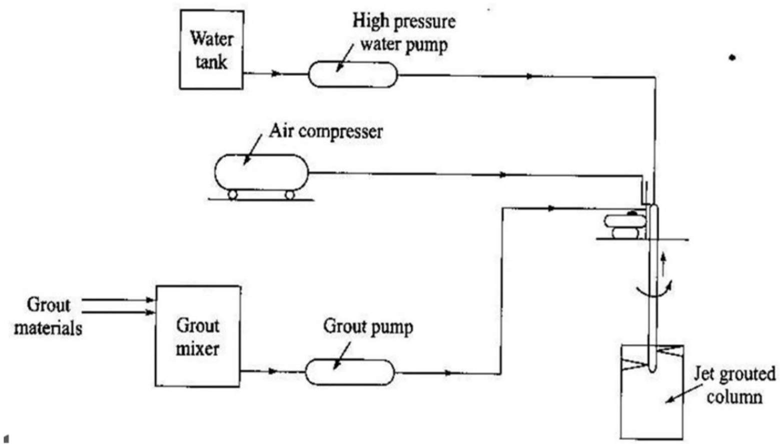
45

Triple Jet Grouting Method



46

Schematic diagram for Triple Jet Grouting



Ground Water Control

Preventing flow either through the sides or into the base of an excavation

Controlling groundwater during tunneling

Preventing or reducing water seepage through a water retention structure such as a dam or flood defence structure

Preventing or reducing contamination flow through the ground

Movement Control

Prevention of ground or structure movement during excavation or tunnelling

Supporting the face or sides of a tunnel during construction or in the long term

Increasing the factor of safety of embankments or cuttings

Providing support to piles or walls to prevent or reduce lateral movement

Support

Underpinning buildings during excavation or tunnelling

Improving the ground to prevent failure through inadequate bearing

Transferring foundation load through weak material to a competent strata

Environmental

Encapsulating contaminants in the ground to

reduce or prevent contamination off site or into

sensitive water systems

Providing lateral or vertical barriers to contaminant flow

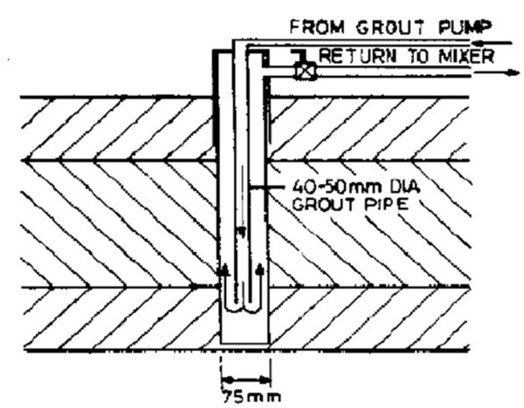
Introducing reactive materials into the ground to treat specific contaminants by creating permeable reactive barriers

Circuit Grouting

Based on the principle of grouting from the top downwards.

A drill hole is bored to the depth of the bottom zone and grout

is pumped down the grout pile and returned up the drill hole.



Point Grouting

In shallow work of 10 to 12 m deep the grout is injected from the points of a driven or jetted lance.

Injections are delivered at pre-determined positions along the line of drive and also on the return in systems where a second reacting grout ingredient is to be placed independently of the initial injection.

Electrokinetic Injection

Stabilization of silty soils may not be possible by chemical or admixture perhaps because of lack of confinement or the necessity to avoid disturbance of the ground.

Chemical stabilizers are introduced at the anode and carried toward the cathode by electro osmosis.

Direct current electrical gradients of the order of 50 to 100 Volts/m are required.

Soil Fracture Grouting

Root-like zones of grout material is formed in the soil mass

Sleeved pipe grouting technique is used

Used for restoration of verticality of a tilted building

SHORING

It is the means of providing support to get stability of a structure temporarily under certain circumstances during construction, repair or alteration.

Such circumstance arises when

1. The stability of a structure is endangered due to removal of a defective portion of the structure.
2. The stability of a structure is endangered due to unequal settlement during construction itself or in long run.
3. Certain alterations are to be done in present structure itself. Eg: remodeling of walls, changing position of windows,etc.
4. Alterations are carried out in adjacent building for remodeling, strengthening of foundation, etc.

INSTALLATION OF SHORING

For shoring timber or steel tubes may be used. Sometimes both are used in combination. If timber is used its surface should be coated with a preservative so as to protect against wet rot.

The shoring should be designed based on the load it has to sustain and duration of load.

Shoring may be given internally or externally depending on the case and in certain cases they may be provided on either side of the wall to produce additional stability.

Shoring should be installed only after getting the permission if necessary, of the local authorities.

There is no time limit to which the shoring has to be kept, it may range from weeks to years depending on the case.

UNDERPINNING

In construction, underpinning is the process of strengthening and stabilizing the foundation of an existing building or other structure. It is process of modifying an existing foundation by adding support.It can also be described as the installation of temporary or permanent support to an existing foundation to provide either additional depth or an increase in bearing capacity.

Whenever a new building is to be built especially in urban areas, it is quite common to have the foundations lower than the foundations of adjacent buildings. It is therefore essential that the stability of the existing building(s) is safeguarded by performing underpinning as well as shoring that care and forethought should be undertaken if these operations are to be successfully carried out. Only highly skilled and experienced personnel or companies should perform these operations.

Reasons for Underpinning

- The original foundation is simply not strong or stable enough.
- The usage of the structure has changed.
- The properties of the soil supporting the foundation may have changed (possibly through subsidence) or were mischaracterized during design.
- The construction of nearby structures necessitates the excavation of soil supporting existing foundations.
- It is more economical, due to land price or otherwise, to work on the present structure's foundation than to build a new one.
- Construction of a new project with a deeper foundation adjacent to an existing building.

Reasons for Underpinning

- To enable the foundations to be deepened for structural reasons e.g to construct a basement or addition of another storey to the building.
- To support a structure that is sinking or tilting due to poor soil or instability of the superstructure.
- As a safeguard against possible settlement of a structure when excavating close to or below its foundation level.
- To support a structure while making alteration to its foundations or main supporting members.
- To increase the width of a foundation to permit heavier loads to be carried e.g when increasing the height of a building with new levels.
- To enable a building to be moved bodily to a new site.
- The original foundation is simply not strong or stable enough.
- Settlement of an existing structure.

The means and methods of supporting a structure foundation depends on some of the following factors:

✓Foundation Loads: static and dynamic, permanent and temporary.

✓Type and magnitude of allowable structural movement i.e deformations.

✓Subsurface soil conditions.

✓Subsurface ground water conditions.

✓Access and mobility to the foundations.

✓Potential for environmental hazards.

✓Seismic loading.

WAYS OF ACHIEVING UNDERPINNING

It can be done by :

i. Load transfer

ii. Soil treatment

iii. A combination of the above two mechanisms

LOAD TRANSFER

This literally take structural loads and transfer them to an underlying stratum that is more suitable for support. Underpinning is accomplished by extending the foundation in depth or in breadth so it either rests on a more supportive soil stratum or distributes its load across a greater area.

SOIL TREATMENT

This changes the physical properties of the ground to make it stronger and more supportive ,often without any change to existing foundations.

In some cases, ground treatment can be utilized to strengthen the ground while also acting as a load transfer.

Use of micropiles and jet grouting are common methods in underpinning. An alternative to underpinning is the strengthening of the soil by the introduction of a grout. All of these processes are generally expensive and elaborate.

59

Types of Underpinning

- Mass Concrete Underpinning
- Beam and base underpinning
- Mini-piled underpinning

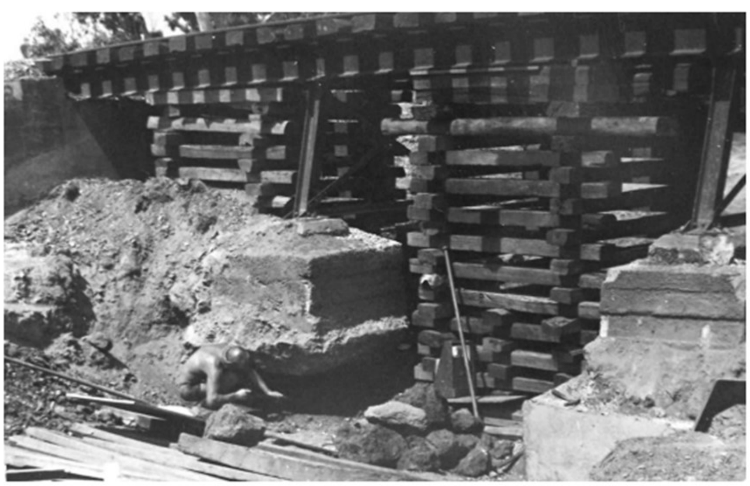
60

Mass Concrete Underpinning

Also known as traditional underpinning. This underpinning method strengthens an existing structure's foundation by digging boxes by hand underneath and sequentially pouring concrete in a strategic order. The final result is basically a foundation built underneath the existing foundation.

This underpinning method is generally applied when the existing foundation is at a shallow depth, however, the method still works very well even at 50 feet deep. The method has not changed since its inception with its use of utilitarian tools such as shovels and post hole diggers. Heavy machinery is not required for this method due to the tight nature of the boxes being dug.

61



Underpinning the foundations of a railway bridge using a timber box crib to support the bridge. A completed concrete pad underpinning can be seen at the bottom right.]

62

Beam and base underpinning

The beam and base method of underpinning is a more technically advanced adaptation. A reinforced concrete beam is constructed below, above or in replacement of the existing footing. The beam then transfers the load of the building to mass concrete bases, which are constructed at designed strategic locations. Base sizes and depths are dependent upon the prevailing ground conditions. Beam design is dependent upon the configuration of the building and the applied loads.

63

Mass Concrete Underpinning

64

Mini-piled underpinning

Mini-piles have the greatest value where ground conditions are very variable, where access is restrictive. Mini-piled underpinning is generally used when the loads from the foundations need to be transferred to stable soils at considerable depths - usually in excess of 5.0 metres. Mini-piles may either be augured or driven steel cased, and are normally between 150 mm and 300 mm in diameter.

65

Left: Mini Piling Auger Rig
Right: Beam and Base construction of beam under existing wall

66

ADVANTAGES OF MICROPILES

- They have high carrying capacity.
- Less site constraint problems.
- Low noise and vibration.
- It is a self –sustained operation.
- It can be designed to have very low settlement.

DISADVANTAGES OF MICROPILES

- Higher cost as compared to other pilling systems.

UNIT-III
MCE-164
Geotechnics of Hill Area

Course Instructor: Dr. V. B. Chauhan

Geosynthetics

2

Content

- Introduction
- Types
- Testing
- Design
- Application in Hilly region

Introduction

- People in ancient times used bamboo, wood and other materials along with sand and mixtures of mud and clay in house construction.
- Geosynthetics includes a family of materials that are used in conjunction with soil to improve its performance.
- Geosynthetic includes- geotextiles, geomembranes, geogrids, geonets, geocomposites, geosynthetic clay liners, geopipes, geobags, etc.
- These are used in road separators; reinforcement of embankments, slopes and retaining walls; moisture/vapors barriers; and erosion control and filtration.
- Henry Vidal introduced the modern concept of reinforced soil in the 1960s.

Types

- Geotextiles
- Geogrids
- Geonets
- Geomembranes
- Pre-fabricated vertical drains (PVD)
- Geosynthetic clay liner (GCL)
- Geocells
- Geocomposites

A Brief Overview of Geosynthetics and Their Major Applications

1. Geosynthetic Materials
2. Transportation and Geotechnical
3. Geoenvironmental
4. Hydraulic Engineering
5. Private Development

1. Geosynthetic Materials

- Polymer Background
- Types of Geosynthetics
- Various Functions
- Design Methods
- Application Areas

Polymer Background

- geosynthetics are really “geopolymers”
- feedstock is natural gas reacted to form resin in a flake form
- mixed with additives into a formulation
- manufactured into a particular type of geosynthetic material

Geosynthetic (GS) Materials

- geotextiles (GT)
- geogrids (GG)
- geonets (GN)
- geomembranes (GM)
- geosynthetic clay liners (GCL)
- geopipe (GP)
- geofoam (GF)
- geocomposites (G C)

Geotextiles (GT)

- majority are made from polypropylene fibers
- standard textile manufacturing
- woven (slit film, monofilament or multifilament)
- nonwoven (needle punched or heat bonded)
- characterized by an open and porous structure
- mechanical and hydraulic properties vary widely
- very versatile in their primary function

Geogrids (GG)

- unitized, woven yarns or bonded straps
- structure allows for soil “strike-through”
- bidirectional – equal strength in both directions
- unidirectional – main strength in machine direction
- focuses entirely on reinforcement applications, e.g.,
- walls, steep slopes, base and foundation reinforcement

Geonets (GN)

- all are made from high density polyethylene
- results in parallel sets of ribs as a integral unit
- biplanar – flow is equal in all directions
- triplanar – flow much greater in machine direction
- function is always in-plane drainage
- surfaces must be covered; usually with GTs

Geomembranes (GM)

- function is always containment
- represents a barrier to liquids and gases
- many types: HDPE, LLDPE, fPP, PVC, EPDM, etc.
- manufactured rolls are field seamed
- required by regulations for waste containment
- new applications in hydraulics and private development

Geosynthetic Clay Liners (GCL)

- function is always containment
- common product is bentonite between 2-GTs
- internally reinforced by needle punched or stitching
- bentonite product bonded to GM is also available
- many other variations exist
- competitive with compacted clay liners (CCLs)
- beneath a GM; one has a composite liner

Geopipe

- its really buried plastic pipe!
- function is always drainage
- HDPE and PVC most common
- both can be smooth walled or corrugated
- corrugated HDPE growth is enormous

Geofoam (GF)

- EPS or XPS in block form
- lightweight fill on soft or sensitive soils
- relieves lateral pressure on walls
- also used for insulation of frost-sensitive soils

Geocomposites (GC)

- array of available products
- GT/GM; GT/GG; GT/GN; etc.
- considerable ongoing innovation
- primary function depends on final product

Function vs. Geosynthetic Type

Type of Geosynthetic	Separation	Reinforcement	Filtration	Drainage	Containment
geotextile	√	√	√	√	
geogrid		√			
geonet				√	
geomembrane					√
geosynthetic clay liner					√
geopipe				√	
geofoam	√				
geocomposite	√	√	√	√	√

Testing

- Tensile strength test
- Puncture resistance
- Trapezoid tearing strength
- Resistance to perforation (Cone drop test)
- Determination of water flow / permeability (normal to the plane)
- Determination of water flow capacity in the plane (transmissivity)
- Cylinder Test
- Evaluation of mechanical damage under repeated loading. Damage caused by granular materials
- Determination of friction characteristics – direct shear test and inclined plane test
- Durability Tests

Application in Hilly region

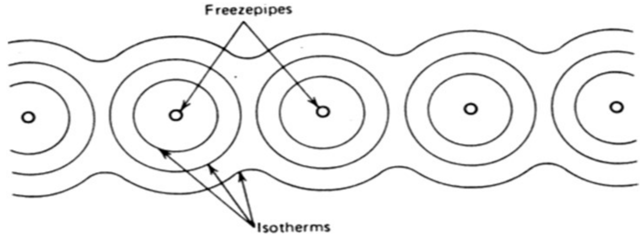
- Hill site development is often related to landslide, and safety of building
- With the recent awareness of risks involved in hill site development, a more proper and systematic control and precaution is taking shape through the private and public sectors.

Frozen Ground

21

Ground Freezing

- **Ground freezing is used for groundwater cutoff, for earth support, for temporary underpinning, for stabilization of earth for tunnel excavation, to arrest landslides and to stabilize abandoned mineshafts.**
- **Typically, a row of freezepipes are placed vertically in the soil, and heat energy is removed through them, in a process remarkably analogous to pumping groundwater from wells.**



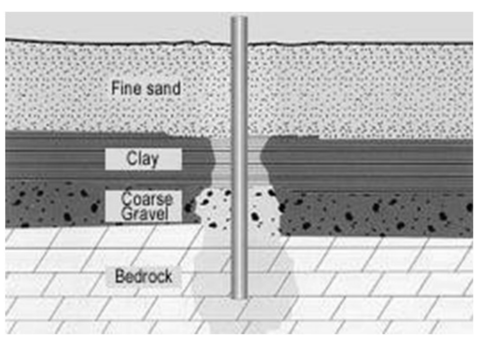
Formation of a freezewall

1 22

- When the earth temperature reaches 32 °F (0 °C), water in the soil pores turns to ice. Then further cooling proceeds.
- With granular soils, the groundwater in the pores freezes readily, and a saturated sand, for example, achieves excellent strength at only a few degrees below the freezing point. Further depression of the temperature produces only marginal increase in strength.
- With clays, however, the ground water is molecularly bonded at least in part to the soil particles.

23

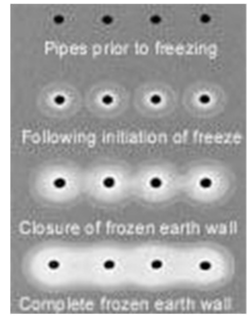
- The design of a frozen earth barrier is governed by the thermal properties of the underlying soils and related response to the freezing system.

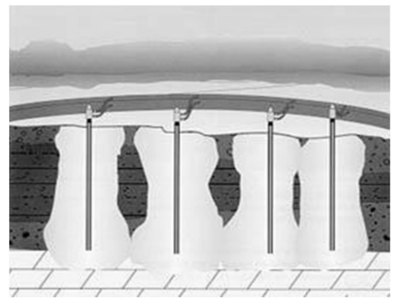


- Formation of frozen earth barrier develops at different rates depending on the thermal and hydraulic properties of each stratum. Typically, rock and coarse-grained soils freeze faster than clays and silts.

24

- When soft clay is cooled to the freezing point, some portion of its pore water begins to freeze and clay begins to stiffen. If the temperature is further reduced, more of the pore water freezes and the strength of the clay markedly increases.
- When designing frozen earth structures in clay it may be necessary to provide for substantially lower temperatures to achieve the required strengths.
- A temperature of +20 °F may be adequate in sands, whereas temperatures as low as –20 °F may be required in soft clay.
- Referring to the figure on slide #1, the frozen earth first forms in the shape of a vertical cylinders surrounding the freezepipes.
- As cylinders gradually enlarge they intersect, forming a continuous wall.





25

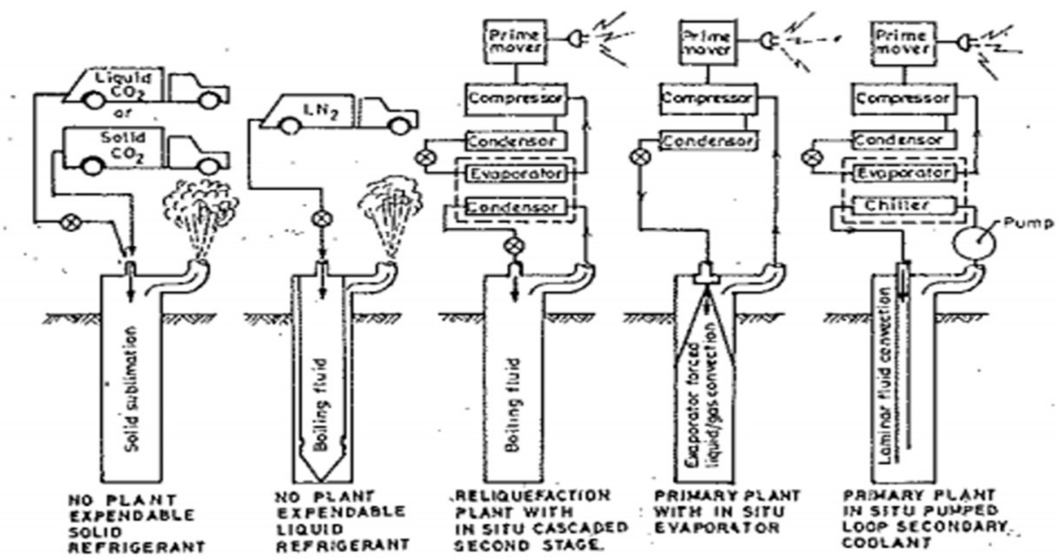
- If the heat extraction is continued at a high rate, the thickness of the frozen wall will expand with time.
- Once the wall has achieved its design thickness, the freeze plant is operated at a reduced rate to remove the heat flowing toward the wall, to maintain the condition.

26

Stabilization by cooling – GROUND FREEZING

- **Reduction in heat increases inter particle repulsion**
- **Freezing of pore water of soil causes thermal stabilization**
- **As freezing begins the oil strength increases considerably as frozen soil is stronger and less pervious**
- **Frozen soil forms non vibration sensitive barrier to seepage flow or soil deformation**
- **For ground water freezing refrigerant is bought to the soil pore water**
- **Pore water moves less than 2 metres per day**
- **As refrigerant pipes begin to freeze all pipes shall start freezing finally making an ice wall**
- **Many schemes are available to provide refrigerant, like Shushter in 1972**
- **Liquid nitrogen, liquid propane, in situ pump methods are some refrigerants.**

27



Refrigeration method adopted from Shushter.

28

Shushter Method

- Freeze pipes are installed at 1m distance
- Liquid refrigerant is injected and allowed to boil
- Frozen zone is often irregular
- Useful for short duration freezing

Pumped loop secondary circulating coolant method

- Useful for freezing over a long time
- Ammonia or freon refrigeration is used
- Many parallel connected freeze pipes with 100 to 200 mm diameter with sealed lower ends
- Coolant is then circulated which is generally Brine.
- Several weeks are needed for effective freezing
- Size and spacing of freeze pipes are important

29

Suggestions for freezing by Koerner 1985

- System is cumbersome to deploy in field
- Once the ice wall is formed the energy required is lesser
- Sands, cohesionless silts and clay can be frozen as long as water in its vicinity is stationary
- Partially dry soil should be pre wetted and then wetted again before freezing
- Accurate drilling of pipes and temperature
- Irregular shape is seen in frozen wall
- Frozen zone is creep sensitive
- Work must be done by a competent contractor only!

30

Blasting techniques

31

Purpose

- Rock excavation is often required on rock foundation projects to remove the material that may not have sufficient bearing capacity, or to form a level bearing surface.
- Blasting is the most common rock excavation method because of its relatively low cost.
- Non-explosive excavation methods- ripping, splitting and the use of hydraulic breakers are suitable where the rock is weak, or there is a need for very precise control of either excavation limits, and/or ground vibration levels.
- The requirements of any excavation method are the use of procedures that break the rock efficiently, while controlling damage to the rock in the bearing surface, of the slopes above and below the foundation, and any nearby structures.

32

Rock fracture by explosives :

Blasting operations comprise the following three tasks:

1. drilling blast holes which have an appropriate diameter, and are laid out in a regular pattern as defined by the burden and spacing;
2. loading the holes with a suitable type and quantity of explosive;
3. detonating the holes in a precise sequence.

- ❑ The design of all these parameters depends on the mechanism by which rock is broken by explosives

33

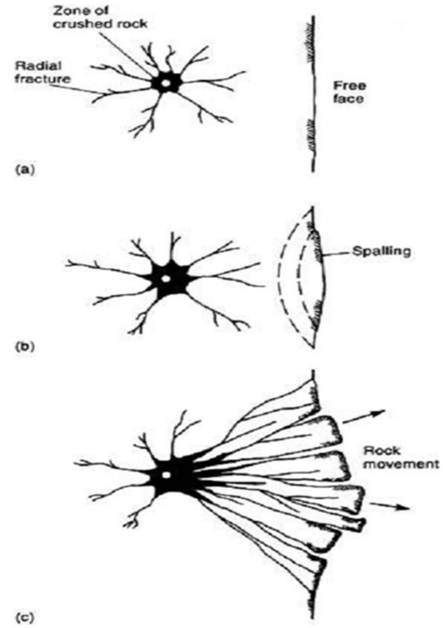
- When an explosive is detonated, it is converted within a few microseconds from a solid to a high temperature gas.
- When confined in a blast hole, this very rapid reaction causes pressures that can reach 100 000 atmospheres to be exerted against the walls of the borehole.
- The explosive energy is transmitted into the rock mass in the form of a shockwave which travels at a velocity of several thousand meters per second.
- Rock breakage, is a three stage process as follows:

1. crushing occurs in the immediate vicinity of the borehole;
2. radial fractures are developed; and
3. movement of the fractured rock mass takes place towards the free face.

34

Mechanism of rock breakage by explosives:

a. crushing and formation of radial fractures;
b. rock spalling on free face; and
c. movement of fractured rock at free face.



- As the shock wave enters the rock surrounding the borehole, the material is crushed in compression for a distance of one to two borehole diameters.
- The expansion of the compressive wave front, the stress level quickly decays below the dynamic compressive strength of the rock.
- At this stage the high gas pressure and the expansion of the borehole develops fractures aligned parallel to the borehole axis in the form of a series of radial cracks that may extend to distances up to 40–50 borehole diameters
- If there is a free face within a distance of about 30 borehole diameters of the hole, a portion of the shock wave is reflected from the face and this results in some spalling of rock on the free face.
- The relief provided by the free face, combined with the force exerted by the expanding high pressure gas, causes movement of the rock that has been weakened and broken into wedge shaped pieces by the formation of the radial cracks.
- This movement of the rock mass extends the radial cracks to the free face resulting in fragmentation of the rock mass.
- This mechanism of rock fracture clearly shows the importance of the presence of a free face, at the correct distance from the blast hole, for efficient blasting operations.

- **The distance between the nearest free face and the blast hole is termed the burden, which is approximately related to the explosive diameter by the following empirical relationship (FHWA, 1985).**

$$B_e = 0.012\left(2\frac{SG_e}{SG_r} + 1.5\right)d_e$$

where B_e is the burden distance in meters; SG_e is the explosive specific gravity; SG_r is the rock specific gravity; and d_e is the explosive diameter in mm.

Preshearing:

- In preshearing, the row of holes along the final face are detonated before the main blast, or on the first delay interval of the main blast.
- This forms a fracture, coincident with the final row of drill holes, which inhibits the extension of the radial cracks from the main blast.
- The row of preshear holes can either be detonated on the same delay, or on separate delays if there is a need to control ground vibrations in the area outside the blast.
- The approximate explosive load per unit length of drill hole to produce a clean presplit line without damage to the wall is given by equation shown on right.

$$w_e = {d_h^2}/{12200} \ (kg/m) \ hole\ dia\ in\ mm$$

where d_h is the drill hole diameter (mm or in). Using this explosive load, the appropriate hole spacing on the preshear line is about 10–12 times the hole diameter.

37

UNIT-IV
MCE-164
Geotechnics of Hill Area

Course Instructor: Dr. V. B. Chauhan

ROCK MASS CLASSIFICATION

1 INTRODUCTION
2 TERZAGHI'S ROCK MASS CLASSIFICATION
3 ROCK QUALITY DESIGNATION (RQD)
4 CLASSIFICATION BASED ON UCS
5 GEOMECHANICS CLASSIFICATION
6 TUNNELLING QUALITY INDEX (Q)
7 ROCK STRUCTURE RATING (RSR)
8 THE GEOLOGICAL STRENGTH INDEX (GSI)
9 ROCK MASS CLASSIFICATION IN SUPPORT DESIGN

Objective of Rock mass classification

- Developed to create in site investigation procedures more systematic and effective
- A common basis for categorizing into different groups
- To identify parameters influencing the behaviour of a rock mass and correspondingly divide the particular rock m
- Improving the quality of site investigations by providing quantitative information for design purposesass formation into groups
- Better engineering judgement and more effective communication on a project

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- Ritter (1879) first attempted to develop a empirical approach for designing tunnel and corresponding support requirement.
- Terzaghi (1946) attempted to categorize rock into different groups based on some descriptive classification and attempted to calculate the rock load on steel sets.
- Deere (1967) successful in providing a quantitative estimate of rock mass quality from drill core logs
- Later: Wickham (1972), Bieniawski (1973, 1989) and Barton et. al (1974)

TERZAGHI'S ROCK MASS CLASSIFICATION

- Rock mass classification for the design of tunnel support.
- Characteristics that dominate rock mass behaviour
- Terzaghi's descriptions:
- Intact rock: contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock.
- Stratified rock: consists of individual strata with little or no resistance against separation along the boundaries between the strata.

- Moderately jointed rock: contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support.
- Blocky and seamy rock: chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked.

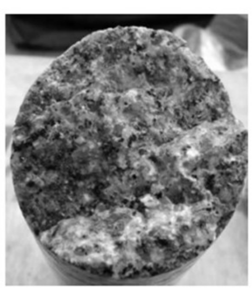
- Crushed but chemically intact rock: fragments are as small as fine sand grains and no re-cementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.
- Squeezing rock: slowly advances into the tunnel without perceptible volume increase. Prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or clay minerals with a low swelling capacity
- Swelling rock: advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks that contain clay minerals such as montmorillonite.

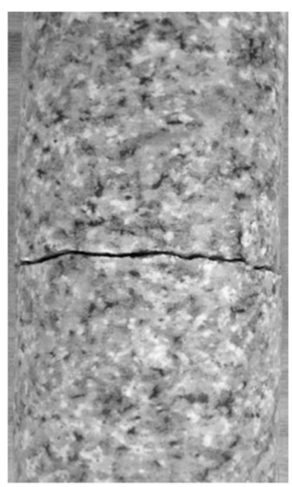
ROCK CORING AND LOGGING

- Details about the rock core drilling and sampling may be found out in ASTM standards (D2113-08) "Standard practice for rock core drilling and sampling of rock for site investigation".
- The corresponding Indian code IS 9179-1979 Indian Standard method for preparation of rock specimen for laboratory testing.

Picture showing core box with cylindrical cores as recovered

Drilling induced fractures





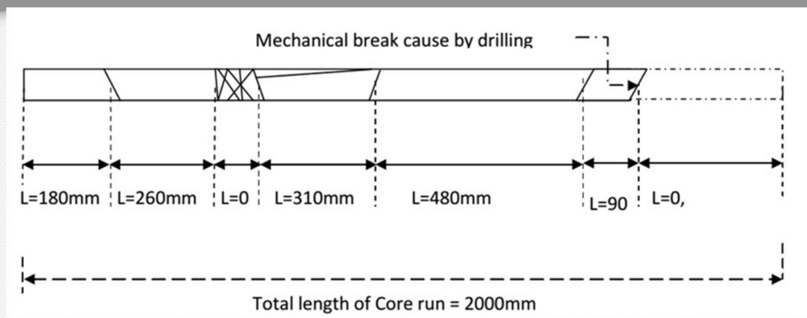
ROCK QUALITY DESIGNATION (RQD)

- Introduced by Deere (1967) to provide a quantitative estimate of rock mass quality from drill core logs.
- Defined: percentage of intact core pieces longer than 100 mm (4 inches) in the total length of core.
- The core should be at least NW size (54.7 mm or 2.15 inches in diameter) and should be drilled with a double-tube core barrel.
- Represent the rock mass quality in situ.

- Palmström (1982):number of discontinuities per unit volume. The suggested relationship for clay-free rock masses.

$$RQD = 115 - 3.3\,J_v$$

- J_v: sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count
- RQD: directionally dependent parameter and its value may change significantly, depending upon the borehole orientation.
- When using diamond drill core, care must be taken to ensure that fractures, which have been caused by handling or the drilling process, are identified and ignored when determining the value of RQD.



There is some difference between natural discontinuities and will normally will be planar, discolored, weathered, usually form in sets and sometimes in filled with clay or gouge material. On the other hand, manmade fractures will normally be irregular and fresh usually random.

$$Core\ Recovery = \frac{\sum Length\ core\ pieces}{Total\ length\ of\ core\ run} \times 100$$

$$Core\ Recovery = \frac{18 + 26 + 31 + 48 + 9}{200} \times 100$$

$$RQD = \frac{\sum(Length\ core\ pieces > 10cm)}{Total\ length\ of\ core\ run} \times 100$$

$$RQD = \frac{18 + 26 + 31 + 48}{200} \times 100 = 61.5\%$$

Rock quality description corresponding to different RQD values

RQD (%)	Rock quality
90-100	Excellent
75-90	Very good
50-75	Good
25-50	Poor
0-25	Very poor

CLASSIFICATION BASED ON UNIAXIAL COMPRESSIVE STRENGTH

Standardization: International Society of Rock Mechanics, ISRM, 1978

Soil	$\sigma_c < 0.25$ MPa
Extremely low strength	$\sigma_c = 0.25 - 1$ MPa
Very low strength	$\sigma_c = 1 - 5$ MPa
Low strength	$\sigma_c = 5 - 25$ MPa
Medium strength	$\sigma_c = 25 - 50$ MPa
High strength	$\sigma_c = 50 - 100$ MPa
Very high strength	$\sigma_c = 100 - 250$ MPa
Extremely high strength	$\sigma_c > 250$ MPa

GEOMECHANICS CLASSIFICATION (RMR)

- Bieniawski (1976): Geomechanics Classification or the Rock Mass Rating (RMR) system
- Based on following parameters
- Uniaxial compressive strength of rock material
- Rock Quality Designation (RQD)
- Spacing of discontinuities
- Condition of discontinuities
- Groundwater conditions
- Orientation of discontinuities

Strength of Intact Rock Material

Rating for the intact rock strength

Point Load Index (MPa)	Uniaxial Compressive strength (MPa)	Rating
>10	>250	15
4-10	100-250	12
2-4	50-100	7
1-2	25-50	4
In this low range only UCS is preferred	5-25	2
	1-5	1
	<1	0

Drill Core Quality – Rock Quality Designation (RQD)

Spacing of discontinuities (mm)	Rating
>2000	20
600-2000	15
200-600	10
60-200	8
<60	5

Spacing of discontinuities

RQD (Percentage)	Rating
90-100	20
75-90	17
50-75	13
25-50	8
<25	3

Condition of discontinuities

Condition of discontinuities	Rating
Very rough surfaces, not continuous, no-separation, un-weathered wall rock	30
Slightly rough surface, separation <1mm, slightly weathered walls	25
Slightly rough surfaces, Separation <1mm, highly weathered walls	20
Slickensided surfaces or gouge <5mm thick or separation 1-5mm continuous	10
Soft gouge >5mm thick or separation >5mm continuous	0

Rating for condition of discontinuity - With detailed information

Discontinuity length (persistence)	Rating	Separation (aperture)	Rating	Roughness	Rating
<1 m	6	None	6	Very rough	6
1-3m	4	<0.1mm	5	Rough	5
3-10m	2	0.1-1.0mm	4	Slightly rough	3
10-20m	1	1-5mm	1	Smooth	1
>20m	0	>5mm	0	Slickensided	0

Rating for condition of discontinuity - Weathering effect

Infilling (gouge)	Rating	Weathering	Rating
None	6	Un-weathered	6
Hard filling < 5mm	4	Slightly weathered	5
Hard filling > 5mm	2	Moderately weathered	3
Soft filling < 5mm	2	Highly weathered	1
Soft filling > 5mm	0	Decomposed	0

Rating for ground water condition

Inflow per 10m tunnel length (litre/min.)	none	<10	10-25	25-125	>125
Joint water pressure / major principal stress	0	0-0.1	0.1-0.2	0.2-0.5	>0.5
General description	completely dry	damp	wet	dripping	flowing
Rating	15	10	7	4	0

Total (Overall) Rock Mass Rating (RMR)

Class Number	Total Rating	Description
I	100-81	Very good rock
II	80-61	Good rock
III	60-41	Fair rock
IV	40-21	Poor rock
V	<21	Very poor rock

Rating Adjustments

Effect of discontinuity strike and dip orientation in tunneling				
Strike perpendicular to tunnel axis	**Drive with Dip (45^o-90^o)**	**Drive with Dip (20^o -45^o)**	**Drive against dip (45^o-90^o)**	**Drive against dip (45^o -90^o)**
	Very favourable	favourable	fair	unfavourable
Strike parallel to tunnel axis	Dip 45^o-90^o	Dip 20^o - 45^o	Dip 0^o-20^o irrespective of strike	
	Very unfavourable	fair	fair	

Rating Adjustment for Discontinuity Orientation

Strike & Dip Orientations	Rating		
	Tunnels & Mines	Foundations	Slopes
Very favourable	0	0	0
Favourable	-2	-2	-5
Fair	-5	-7	-25
Unfavourable	-10	-15	-50
Very Unfavourable	-12	-25	--

TUNNELLING QUALITY INDEX (Q)

- Rock Tunnelling Quality Index, Q (or Norwegian Q system)
- Discovered by Barton, Lien and Lunde (1974)
- Norwegian Geotechnical Institute (NGI)
- Based on 200 case histories of tunnels and caverns

$$Q = \left(\frac{RQD}{Jn}\right) \times \left(\frac{Jr}{Ja}\right) \times \left(\frac{Jw}{SRF}\right)$$

RQD = Rock Quality Designation	100 - 10
J_n = Joint set number	1 – 20
J_r = Joint roughness factor	4 -1
J_a = Joint alteration and clay fillings	1 – 20
J_w = Joint water inflow or pressure	1 – 0.1
SRF = stress reduction factor	1 – 20

Typically: $0.01 < Q < 100$

- (RQD/J_n) = crude measure of block size
- (J_r/J_a) = roughness/friction of surfaces
- (Jw/SRF) = ratio of two stress parameters (active stress)

- (J_r/J_a) = roughness and frictional characteristics of the joint walls or filling materials (favour of rough, unaltered joints in direct contact): such surfaces will be close to peak strength, and will dilate strongly when sheared making it favourable to tunnel stability.
- (RQD/J_n) =structure of the rock mass, is a crude measure of the block or particle size

- Tip: When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly. Where no rock wall contact exists, the conditions are extremely unfavourable to tunnel stability.

- ((J_w/SRF) of Two major parameters
- SRF (total stress parameter) is measure of
- 1) loosening load in the case of an excavation through shear zones and clay bearing rock
- 2) rock stress in competent rock
- 3) squeezing loads in plastic incompetent rocks

- = wate J_w ressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may, in addition, cause softening and possible outwash in the case of clay-filled joints.

Classification of individual parameters used in the Tunnelling Quality Index (Q)

Description	Value
1.Rock quality designation	RQD
2.Very poor	0-25
3.Poor	25-50
4.Fair	50-75
5.Good	75-90
6.Excellent	90-100

Notes: 1.Where RQD is reported or measured as ≤10 (including 0)
2.RQD intervals of 5,i.e.100,95,90 etc. are sufficiently accurate

Index value for joint set number

2. Joint set number	Jn
A. massive, no or few joints	0.5-1.0
B. one joint set	2
C. One joint set +Random	3
D. Two joint set	4
E. Two joint set +random	6
F. Three joint set	9
G. Three joint set+ random	12
H. Four or more joint set, random Heavily jointed, 'sugar cube' etc	15
J. Crushed rock, earthlike	20

Notes: 1. For intersection use (3.0*Jn), 2. For portals use (2.0*Jn)

Index value for joint roughness number

3.Joint Roughness number	Jr
a. Rock wall contact	
b. Rock wall contact before 10cm shear	
A. Discontinuous joints	4
B. rough and regular, undulating	3
C. Smoot undulating	2
D. Slickensided undulating	1.5
E. Rough or irregular, planar	1.5
F. Smooth, planar	1.0
G. Slickensided, planar	0.5
c. no rock wall contact when shared	
H. zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)

1. Add 1.0 if the mean spacing of relevant joint set is greater than 3m.

Index value for joint water reduction number

5. Joint water reduction	J_w	Approx water reduction pressure(kgf/cm^2)
A.Dry excavation or minor inflow i.e < 5l/m locally	1.0	<1.0
B. Medium inflow or pressure, occasional outwash of joint filling	0.66	1.0-2.5
C. large inflow or high pressure in competent rock with unfilled joints	0.5	2.5-10.0
D large inflow or high pressure	0.33	2.5-10.0
E. Exceptionally high inflow or pressure at blasting decaying with time	0.2-0.1>10	
F. Exceptionally high inflow or pressure	0.1-0.05	>10

Notes: 1. Factors C to F are crude estimates increases Jw if drainage installed
2. Special problems caused by ice formation are not considered

Index value for stress reduction factor

6. Stress reduction factors	SRF
a. weakness zones intersecting exaction, which may cause loosening of rock mass when tunnel is excavated	
A. multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth	10.0
B. singles weakness zones containing clay , or chemically disintegrated rock(excavation depth<50m)	5.0
C. single weakness zones containing clay ,or chemically disintegrated rock(excavation depth>50m)	2.5
D. multiple shear zone in competent rock (clay free),loose surrounding rock(any depth)	7.5
E. single shear zone in competent rock(clay free).(depth of excavation<50m)	5.0
F. single shear zone in competent rock(clay free).(depth of excavation>50m)	2.5
G. loose open joints, heavily jointed or sugar cube (any depth)	5.0

Index value for stress reduction factor

b. competent rock, rock stress problems			
	σ_c / σ_1	σ_t / σ_1	SRF
H. low stress, near surface	>200	>13	2.5
I. Medium stress	200-10	13-0.66	1.0
J. high stress, very tight structure(usually favourable to wall stability, may be unfavourable to wall stability)	10-5	0.66-0.33	0.5-2
K. mild rockburst(massive rock)	5-2.5	0.33-0.16	5-10
L. heavy rockburst(massive rock)	<2.5	0.16	10-20

Index value for stress reduction factor

c. squeezing rock, plastic flow of incompetent rock under influence of high rock pressure	
M. mild squeezing rock pressure	5-10
N. heavy squeezing rock pressure	10-20
d. swelling rock, chemical swelling activity depending on presence of water	
O. Mild swelling rock pressure	5-10
P. Heavy swelling rock pressure	10-15

ROCK STRUCTURE RATING (RSR)

- Wickham (1972)
- Quantitative method for describing the quality of support mass and selecting appropriate support on the basis of their rock structure rating on the case studies of small tunnels supported with steel sets

$$RSR = A + B + C$$

- Parameter A: Geology - General appraisal of geological structures on the basis of, rock type and its origin, their hardness and strength, and its geologic structure.
- Parameter B: Geometry - effect of discontinuity pattern with respect to the direction of the tunnel drive on the basis of joint spacing, orientation and direction of tunnel drive.
- Parameter C: Effect of ground water inflow and joint condition on the basis of overall quality on the basis of A & B combined, joint condition and amount of water inflow.

Rock structure rating: parameter A: general area geology

	Basic rock type			
	Hard	Medium	Soft	Decomposed
Igneous	1	2	3	4
Metamorphic	1	2	3	4
Sedimentary	2	3	4	4

	Geological structure			
Igneous		Slightly	moderately	intensively
Metamorphic		Folded or	Folded or	Folded or
Sedimentary	Massive	Faulted	Faulted	Faulted
Type 1	30	22	15	9
Type 2	27	20	13	8
Type 3	24	18	12	7
Type 4	19	15	10	6

Rock structure rating: parameter B: Joint pattern, direction of drive

	Strike perpendicular to axis				
	Direction of drive				
	Both	With dip		Against dip	
	Dip of prominent joints [a]				
Average joint spacing	Flat	Dipping	Vertical	Dipping	Vertical
1.Very closely jointed <2 in	9	11	13	10	12
2.Closely jointed,2-6 in	13	16	19	15	17
3.Moderately jointed , 6-12 in	23	24	28	19	22
4.moderate to blocky,1-2 ft	30	32	36	25	28
5.Blocky to massive,2-4 ft	36	38	40	33	35
6.Massive,>4ft	40	43	45	37	40

Rock structure rating: parameter B: Joint pattern, direction of drive

	Strike parallel to axis		
	Direction of drive Either direction		
	Dip of prominent joints		
Average joint spacing	Flat	Dipping	Vertical
1.Very closely jointed <2 in	9	9	7
2.Closely jointed,2-6 in	14	14	11
3.Moderately jointed , 6-12 in	23	23	19
4.moderate to blocky,1-2 ft	30	28	24
5.Blocky to massive,2-4 ft	36	24	28
6.Massive,>4ft	40	38	34

Rock structure Rating: parameter C: groundwater, joint condition

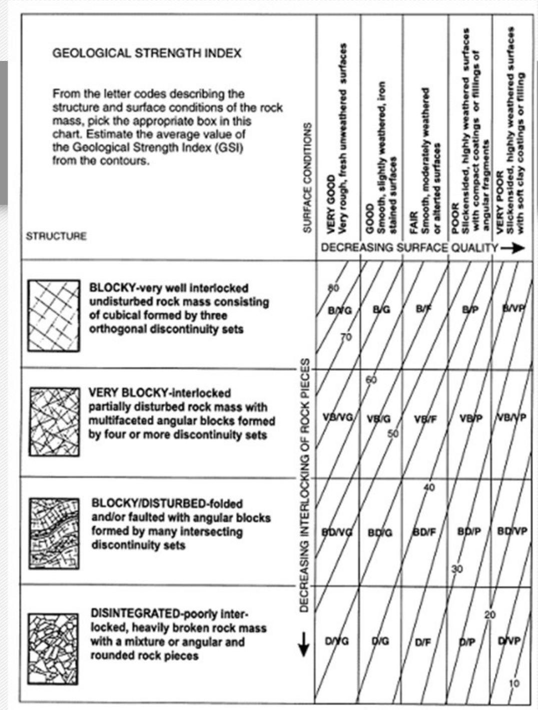
	Sum of parameter A and B					
	13-44			45-75		
Anticipated water inflow	Joint condition [b]					
Gpm/1000ft of tunnel	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight,<200gpm	19	15	9	23	19	14
Moderate,200-1000 gpm	15	22	7	21	16	12
Heavy,>1000gpm	10	8	6	18	14	10

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

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

THE GEOLOGICAL STRENGTH INDEX (GSI)

- Proposed in 1995
- used for the estimation of the rock mass strength and the rock mass deformation modulus
- concentrates on the description of two factors, rock structure and block surface conditions
- The guidelines given by the GSI system are for the estimation of the peak strength parameters of jointed rock masses



ROCK MASS CLASSIFICATION IN SUPPORT DESIGN

Underground supports design corresponding to RMR

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock *RMR*: 81-100	Full face, 3 m advance.	Generally no support required except spot bolting.		
II - Good rock *RMR*: 61-80	Full face , 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock *RMR*: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None.
IV - Poor rock *RMR*: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor rock *RMR*: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

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

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

- ESR is related to the intended use of the excavation and to the degree of security which is demanded of the support system installed to maintain the stability of the excavation.

ESR value for different excavations category

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

- Barton et al (1980) provide additional information on rockbolt length, maximum unsupported spans and roof support pressures to supplement the support recommendations. The length L of rockbolts can be estimated from the excavation width B and the Excavation Support Ratio

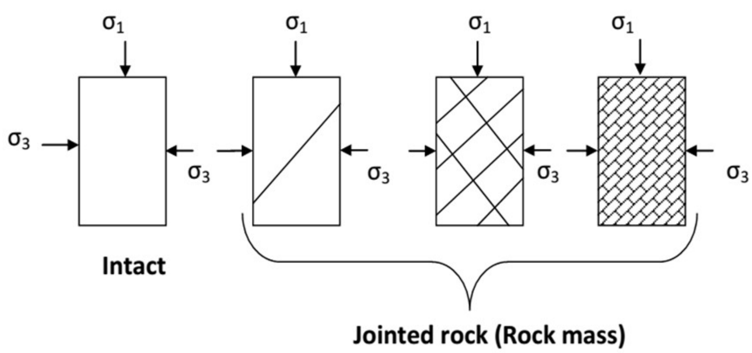
$$L = 2 + \frac{0.15B}{ESR}$$

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

- Based upon analyses of case records, Grimstad and Barton (1993) suggest that the relationship between the value of Q and the permanent roof support pressure P_{roof} is estimated from

$$P_{roof} = \frac{2\sqrt{Jn}\,Q^{-\frac{1}{3}}}{3Jr}$$

Description of intact rock and rock mass



A typical view of rockmass encountered in the field

ROCK ENGINEERING PROBLEMS AND APPLICATION AREAS

- Application includes in many different disciplines like Civil engineering, Mining engineering, Petroleum engineering, Geology and Geophysics.
- A particularly important event in the development of the subject was the merging of elastic theory, with the discontinumm approach.

List of some physical and mechanical properties of rocks

PHYSICO MECHANICAL PROPERTIES	
Physical Properties	**Mechanical Properties**
Mineralogical composition - mineral structure, texture.	Elastic Modulus/ Deformation modulus and Poisson's ratio
Specific gravity, density, unit weight	Uniaxial compressive strength
Porosity, void ratio	Tensile strength
Moisture content, degree of saturation	Shear strength Properties
Permeability	Point load strength
Swelling properties	Rock hardness
Anisotropy	
Electrical properties	
Thermal properties	
Velocity of Elastic waves	
Durability	

PHYSICAL PROPERY

- Density
- Unit weight
- Specific gravity
- Water content

- Highly porous rocks and relatively poor arrangement of grains (less packing) usually have relatively less densities and vice versa.
- The bulk unit weight considers the bulk (total) volume of rocks where as the solid unit weight considers volume excluding the pores, fissures.
- Bulk unit weight depends on the type of rock, its porosity and geological processes that take place in it.

Density, $\rho_s = \frac{Mass\ of\ solid}{Volume} kg/m^3$

Unit weight of solid, $\gamma_s = \frac{Weight\,(G)\,of\ dry\ rock\ sample}{True\ volume\,(V)} kg/m^3$

True volume signifies the grains without pores and fissures included.

Bulk unit weight, $\gamma_o = \frac{Weight\,(G)\,of\ dry\ rock\ sample}{Volume\,(V_o)\,of\ the\ skeleton\,(including\ pores\ and\ fissures)} kg/m^3$

Average bulk unit weight for some common rocks

Rock	**Average Bulk unit weight (kN/m^3)**
Granite	27
Basalt	30
Gneiss	27
Marble	27
Schist	26
Sandstone	26
Hard coal	15

The water content of rock specimen can be calculated directly by dividing mass of pore water to mass of sample. For the determination of porosity and density saturation and caliper technique, saturation and buoyancy, mercury displacement and grain specific gravity technique are usually followed.

Saturation and Buoyancy technique

Applicable only to non-friable coherent rocks that can be machined and rocks that do not swell or disintegrate when they are oven dried or when immersed in water. At least three specimens selected such that minimum size should be of mass 50g or minimum dimension should be ten times greater than maximum grain size whichever is greater.

Apparatus required:

Oven, Desiccator,
Vernier, Vacuum saturation equipment,
Sample container, Balance,
Immersion bath and Wire basket.

Procedure:

- The sample is washed in water to remove dust and then is saturated in water for 1 hour with a vacuum pressure of 0.8 kPa
- Determine the mass of wire basket submerged into immersion bath, M1
- Transfer the mass of sample into wire basket into immersion bath and determine the mass. M2
- Determine the mass of container which should be in clean and dry with lid, M3
- Remove the sample from immersion bath and surface dry it with moist cloth. Place the sample into the container with lid and determine their mass, M4
- Take out the lid and place the sample with container into the oven @ 105°C for 24 hours
- Place the sample in desiccators and allow it cool for 30 minutes
- Determine the mass of dry sample with container provided with lid, M5

Calculations

Saturated-Submerged mass, $M_{sub} = M_2 - M_1$ (kg)

Saturated-Surface dry mass, $M_{sat} = M_4 - M_3$ (kg)

Dry mass, $M_s = M_5 - M_3$ (kg)

Bulk volume, $V = \frac{M_{sat} - M_{sub}}{\rho_w}$ (m^3)

Pore volume, $V_v = \frac{M_{sat} - M_{sub}}{\rho_w}$ (m^3)

Porosity, $n = \frac{V_v}{V} \times 100$ (%)

Dry density, $\rho_d = \frac{M_s}{V}$ (kg/m^3)

Relative density, $G_m = \frac{\rho}{\rho_w}$

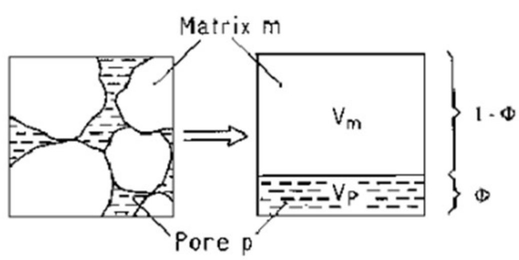
Porosity

Rocks contain voids in the form of pores, joints (fissures) etc. The voids may be inter connected or separated from one another. If they are inter connected and pressure gradient exists – rock can conduct fluids or gases. Porosity is an intrinsic property and is the ratio of the volume of openings (voids) to the total volume of material.

$$\text{The Porosity 'n'} = \frac{V_v \text{ (Pore volume)}}{V_o \text{ (Bulk volume)}} \quad \text{in \%}$$

Porosity represents the storage capacity of the geologic material.
The primary porosity of a sediment or rock consists of the spaces between the grains that make up that material. The more tightly packed the grains are, the lower the porosity.
The porosity of the box of marbles would be determined by dividing the total void space by the total volume of the sample and expressed as a percentage. The primary porosity of unconsolidated sediments is determined by the shape of the grains and the range of grain sizes present. In poorly sorted sediments, those with a larger range of grain sizes, the finer grains tend to fill the spaces between the larger grains, resulting in lower porosity. Primary porosity can range from less than one percent in crystalline rocks like granite to over 55% in some soils. The porosity of some rock is increased through fractures or solution of the material itself. This is known as secondary porosity.
Usually igneous or metamorphic rocks will have very low porosity (0-2%) where as sedimentary rocks like sandstones will have very high porosity (upto 40%). Many factors which affect porosity like, grain size distribution, grain shape and arrangement, degree of cementation of grains, applied pressure etc. Porosity decreases with increase of pressure and therefore, deep seated deposit with large overlying pressure may tend to have relatively low porosity compare to surface depositions.

Visual representation of a porous rock



High porosity	Highly porous marine sediments
	Unconsolidated sediments
	Sandstones
	Carbonate rocks
Low porosity	Fractured igneous rocks, other dense rock types

Porosity may be represented with void index which can be found using quick absorption technique.
Void index defined as the mass of water contained in a rock sample after one hour period of immersion, as a percentage of its initial dry mass.
The index is correlated with porosity and also with such properties as degree of weathering or alteration.
The test should only be used for rocks that do not appreciably disintegrate when immersed in water.
The void index is evaluated from the ratio of difference between the saturated and dry weight of rock to dry weight of rock expressed in terms of percentage.

Permeability

- measure of the ease with which fluids will flow though a porous rock and is also an intrinsic property.
- Permeability Controlling factor: packing, shape, and sorting of granular materials.
- Although a rock may be highly porous, if the voids are not interconnected, then fluids within the closed, isolated pores cannot move. The degree to which pores within the material are interconnected is known as effective porosity.
- Rocks such as pumice and shale can have high porosity, yet can be nearly impermeable due to the poorly interconnected voids.
- Well-sorted sandstone closely replicates the example of a box of marbles cited above. The rounded sand grains provide ample, unrestricted void spaces that are free from smaller grains and are very well linked.
- Consequently, sandstones of this type have both high porosity and high permeability.

- Permeability is part of the proportionality constant in Darcy's law which relates discharge (flow rate) and fluid physical properties (e.g. viscosity), to a pressure gradient applied to the porous media.

$$v = \frac{\kappa}{\mu}\frac{\Delta P}{\Delta x} \qquad \kappa = v\frac{\mu \Delta x}{\Delta P}$$

- v = superficial fluid flow velocity through the medium (i.e., the average velocity calculated as if the fluid were the only phase present in the porous medium) (m/s)
- κ = permeability of a medium (m^2)
- μ = dynamic viscosity of the fluid (Pa.s)
- ΔP = applied pressure difference (Pa)
- Δx = thickness of the bed of the porous medium (m)
- Permeability in rocks depends on the number and kind of pores and joints present, pressure of water, direction etc.

Permeability coefficient for some common rock types

Rock type (intact)	K_γ (cm/s)
Sanstone	0.2-6 X 10^{-9}
Granite	0.5-2 X 10^{-10}
Limestone	1-12 X 10^{-12}
Schist	0.5-1.5 X 10^{-10}

The proportionality constant specifically for the flow of water through a porous media is called the hydraulic conductivity; permeability is a portion of this, and is a property of the porous media only, not the fluid.

STRESSES AROUND UNDERGROUND OPENINGS

Underground openings: Applications

- Tunnels built for highways and railroads
- Water supply and sewage tunnels
- Underground power stations
- Storage caverns

It is necessary to understand the various aspects of underground openings and their stress and deformation characteristics.

Rocks are initially stressed and any opening created cause a changes in the initial stress.

The post excavation state of stress in the structure is the resultant of initial state of stresses and stresses induced by excavation.

Hence the determination of the state of stress is necessary for any design analysis.

The study of stresses around underground openings gives an insight into the basic mechanisms like displacements and the stress fields and helps to provide suitable support for the underground opening. The major conditions around an opening can be classified as in-situ stresses– due to the overburden rock, induced stresses– due to the excavation for the opening and traffic loads– not significant in the case of deep tunnels.

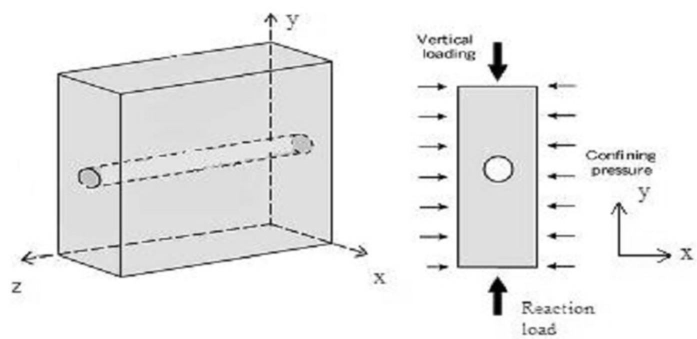
Different type of tunnels and underground excavation include road tunnels, rail tunnels, rapid transit tunnels, water tunnels, sewage tunnels, hydroelectric tunnels, service and utilities tunnels, station buildings etc.

Rock excavation can be made adopting any of the following:

- Drilling and blasting
- Using tunnel boring machines (TBM): Very Popular
- Road headers
- Sequential excavation with small mechanical equipments

STRESSES AROUND UNDERGROUND OPENING

These 3D problem can be reduced to a 2D if the stresses on the periphery of a long tunnel are studied by the sectional analysis. The problem is reduced to the study of the stresses, on the periphery of an opening in a thin sheet of elastic or elasto-plastic medium.



Sectional view of a long tunnel

- To transform the stresses from rectangular co-ordinate system to polar co-ordinate system the following transformation equations are used.

$$\sigma_r = \frac{\sigma_x + \sigma_y}{2} - \frac{\sigma_x - \sigma_y}{2}\cos 2\theta + \tau_{xy}\sin 2\theta$$

$$\sigma_\theta = \frac{\sigma_x + \sigma_y}{2} + \frac{\sigma_x - \sigma_y}{2}\cos 2\theta - \tau_{xy}\sin 2\theta$$

$$\tau_{r\theta} = \tau_{xy}\cos 2\theta - \frac{\sigma_x - \sigma_y}{2}\sin 2\theta$$

The back formulations of horizontal and vertical stresses are given by

$$\sigma_x = \frac{\sigma_\theta + \sigma_r}{2} - \frac{\sigma_\theta - \sigma_r}{2}\cos 2\theta - \tau_{xy}\sin 2\theta$$

$$\sigma_y = \frac{\sigma_\theta + \sigma_r}{2} + \frac{\sigma_\theta - \sigma_r}{2}\cos 2\theta + \tau_{xy}\sin 2\theta$$

$$\tau_{xy} = \tau_{r\theta}\cos 2\theta - \frac{\sigma_\theta - \sigma_r}{2}\sin 2\theta$$

- Considering an infinite plate of thickness 't' with a circular hole of radius 'a' at the origin. If the applied stresses in the horizontal (x) and vertical (z) directions are σ_x & σ_z respectively, the stress components are

$$\sigma_\theta = \frac{1}{2}(\sigma_x + \sigma_z)\left(1 + \frac{a^2}{r^2}\right) - \frac{1}{2}(\sigma_x - \sigma_z)\left(1 + \frac{3a^4}{r^4}\right)\cos 2\theta$$

$$\sigma_r = \frac{1}{2}(\sigma_x + \sigma_z)\left(1 - \frac{a^2}{r^2}\right) + \frac{1}{2}(\sigma_x - \sigma_z)\left(1 + \frac{3a^4}{r^4} - \frac{4a^2}{r^2}\right)\cos 2\theta$$

$$\tau_{r\theta} = -\frac{1}{2}(\sigma_x - \sigma_z)\left(1 - \frac{3a^4}{r^4} + \frac{2a^2}{r^2}\right)\sin 2\theta$$

a = radius of the circular opening

θ = central angle with x-axis

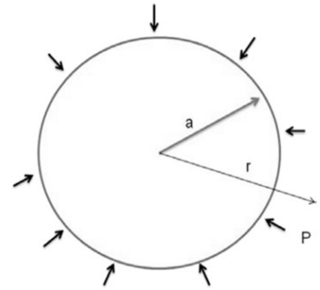
r = radial distance of the element from the center of the opening

- For $\sigma_x = 0$, the maximum tangential stress is three times the applied stress and occurs at the boundary on the X-axis, that is θ = 0 or π .
- θ = π/2 or 3π/2, the tangential stress at the boundary of the opening is equal to the applied stress but is of opposite in sign.
- **Case 1: Hydrostatic case When, $\sigma_x = \sigma_z = p$ (compressive), i.e. for the case of hydrostatic loading:**

$$\sigma_r = p\left(1 - \frac{a^2}{r^2}\right)$$

$$\sigma_\theta = p\left(1 + \frac{a^2}{r^2}\right)$$

$$\tau_{r\theta} = 0$$

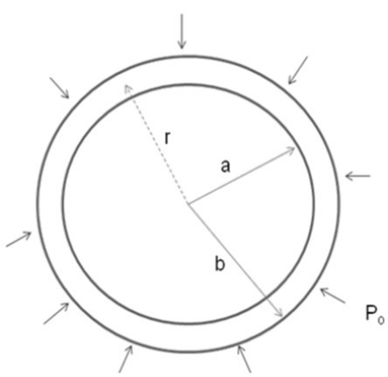


Circular opening in hydrostatic stress field

- If $\sigma_x = \sigma_z$, the maximum tangential stress occurs at the boundary of circular opening and is equal to two times the applied stress. The radial stress at the boundary is equal to the two times of applied stress and occurs on a plane at 45^o to the boundary.

Thick wall cylinder subjected external pressure

- This problem corresponds to the problem of a tunnel or shaft lining (with a internal and external radius 'a' and 'b' respectively) in a rock formation having a hydrostatic stress field (P).



$$\sigma_r = -\frac{b^2 P_o}{b^2 - a^2}(1 - \frac{a^2}{r^2})$$

$$\sigma_\theta = -\frac{b^2 P_o}{b^2 - a^2}(1 + \frac{a^2}{r^2})$$

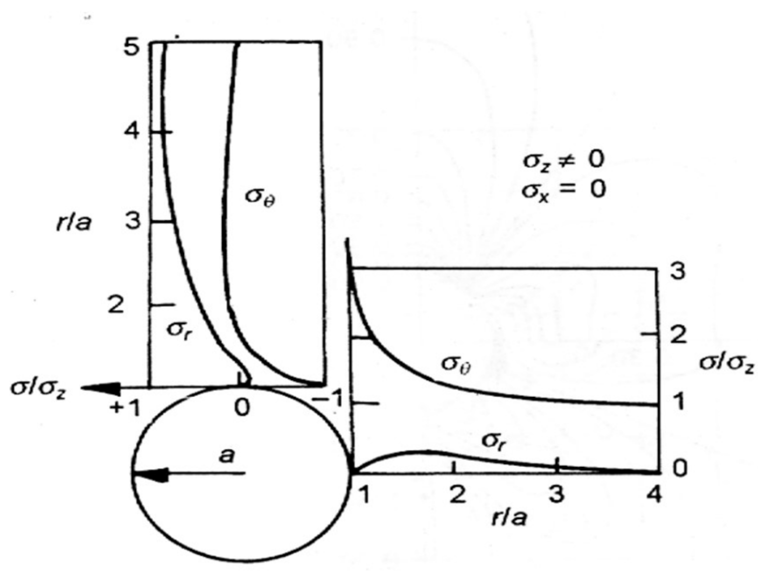
$$\tau_{r\theta} = 0$$

Tunnel linings in plain strain with internal radius 'a' and external radius 'b' in hydrostatic stress field

If one takes a limit for b→ infinity , it can be seen that , the results are same as circular opening in hydrostatic stress field.

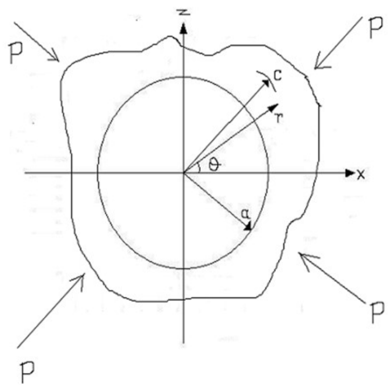
CASE 2: When $\sigma_x = 0$

- For $\sigma_x = 0$, the maximum tangential stress is three times the applied stress and occurs at the boundary on the X-axis that is $\theta = 0$ or π.
- When $\theta = \pi/2$ and $3\pi/2$, the tangential stress at the boundary of the opening is equal to the applied stress but is of opposite in sign.



Tangential and radial stresses around circular openings when σ_x=0

CIRCULAR HOLE IN AN ELASTO-PLASTIC INFINITE MEDIUM UNDER HYDROSTATIC LOADING



Circular hole in an elasto-plastic infinite medium under hydrostatic loading

Using Tresca's yield criteria

- For $a \le r \le c$

$$\sigma_{rp} = 2hp \ln \frac{r}{a}$$

$$\sigma_{\theta p} = 2hp\left(1 + \ln \frac{r}{a}\right)$$

- For $r \ge c$

$$\sigma_r = p\left[1 - \frac{ha^2}{r^2} e^{(1-h)/h}\right]$$

$$\sigma_\theta = p\left[1 + \frac{ha^2}{r^2} e^{(1-h)/h}\right]$$

σ_r, σ_{rp} = radial stress in elastic and plastic zone respectively

σ_θ, $\sigma_{\theta p}$ = tangential stress in elastic and plastic zone respectively

a = radius of the circular opening

θ = central angle with x-axis

r = radial distance of the element from the center of the opening

$c = ae^{(1-h)/2h}$ = radius of boundary between the elastic and plastic zones

p = applied hydrostatic pressure, compressive

h = k / p.

K= stress ratio= ratio of tangential stress and radial stress (values will be provided)

Elasto- Plastic Observations

- The tangential stresses at the boundary of cylindrical openings are considerably lower for the elasto-plastic rock mass than for a perfectly elastic one.
- The tangential stresses beyond the plastic zone are larger than the perfectly elastic case at the same radial distance
- The zone of influence due to opening is larger than that in the case of perfectly elastic rock.

ZONE OF INFLUENCE

Zone of influence of an excavation is very important for underground tunneling and mining applications where multiple excavation/ tunnels are excavated. With considerable simplification of a design problem, idea is to get the domain of significant disturbance of the excavation stress, and get the stresses near field and far field of an opening. Stress distribution around a circular hole in hydrostatic medium.

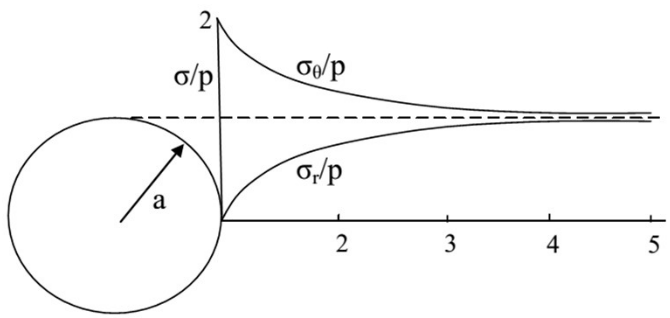
$$\sigma_r = p\left(1 - \frac{a^2}{r^2}\right) \qquad r = 5a, \ \sigma_\theta = 1.04p \qquad \sigma_r = 0.96p$$

$$\sigma_\theta = p\left(1 + \frac{a^2}{r^2}\right)$$

$$\tau_{r\theta} = 0$$

At r = 5a, the state of stress is not significantly different (within 5%) from the field stress

Radial and tangential stresses corresponding to different radial distance in a hydrostatic stress field



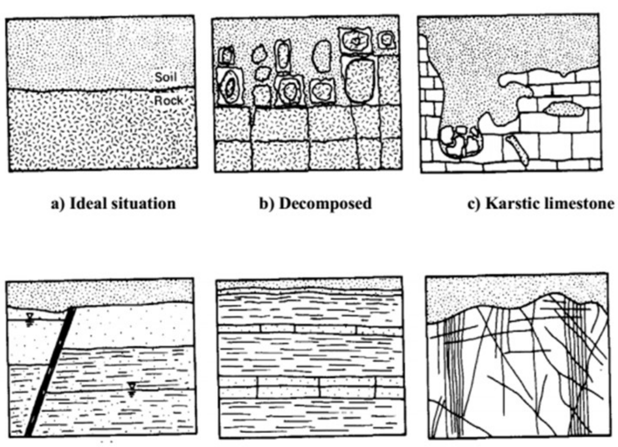
Tangential and radial stresses corresponding to different radial distance in hydrostatic stress field

Distance from centre (r)	Tangential stress (σ_θ)	Radial stress (σ_r)
a	2P	0
2a	1.25P	0.75P
3a	1.11P	0.88P
4a	1.06P	0.94P
5a	1.04P	0.96P

FOUNDATION ON ROCKS

- Usually rocks are considered to be a **good foundation material and strength** is rarely an issue while designing any structure over rock. But, inherently rocks comes with weaknesses and that comes the real challenges for the design. Moreover, if the load is really very large such as in cases of heavy bridge piers or sky scrapers etc., the bearing capacity may need to be checked specially for rocks which rocks is weak or moderately strong.
- Sometimes, rocks may have defects, or inherently weak like chalks, clay shales or clay bands, friable sandstones, tuffs or porous limestones, or sometimes rocks may be highly weathered or fractured, such situations heavy foundation load may develop excessive deformation which is undesirable.
- There are issues with the rock foundations specially if the rocks are decomposed, in karstic limestone which are soluble, rocks with faults, weathered rock with or without filling material, fractured/ fissured rocks etc.

Different problematic rocks



a) Ideal situation b) Decomposed c) Karstic limestone

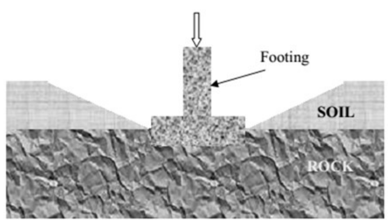
d) Faulted rock e) Weathered rock with residual soil f) Fractured rock

FOUNDATION TYPES ON ROCKS

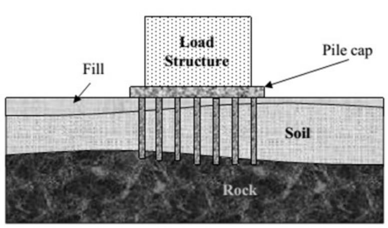
- Shallow foundation
- Deep/Pile foundation
- Rock socketed Piers

Failure in rock foundations may happen in a number of modes. Mode of hard, brittle rock may be totally different than the mode observed from weak rocks. More ever, if joints/fissures are present, same rock may again have a totally different mode of failure.

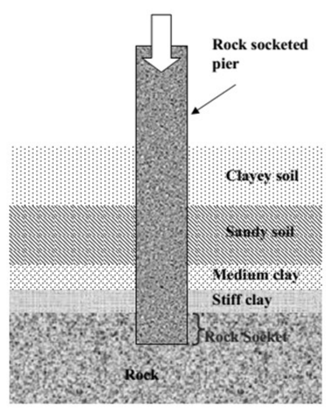
Rocks are weak in tension and therefore the propagation of extension cracks leads to indentation of the loaded foundation on rock. Once the load reaches the tensile strength of the rock crack initiates, further loading may extends the crack and with still higher load, cracks coalesce and interfere leading to eventual failure. Foundation on rock masses undergoes additional permanent deformation due to the closure of the fissures, pores and cracks.



(a) Footing on rock



(b) End bearing piles resting on rock

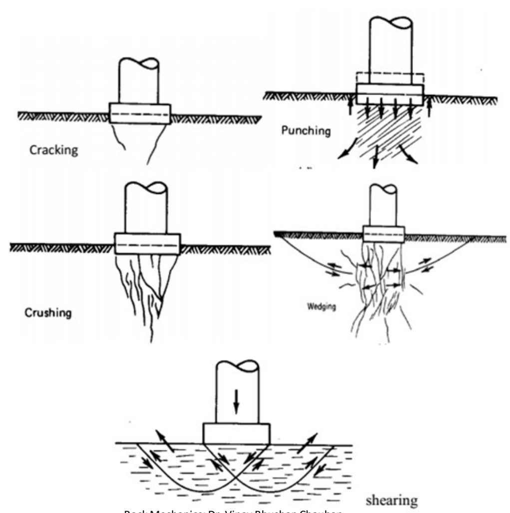


c) Rock socketed pier

BEARING CAPACITY

- The bearing capacity of foundations founded on rock masses depends mostly on the ratio of joint spacing to foundation width, as well as intact and rock mass qualities like joint orientation, joint condition (open or closed), rock type, and intact and mass rock strengths.
- Two main characteristics: They have to be safe against overall shear failure in the rock mass that supports them. They cannot undergo excessive displacement, or settlement. Hence, there are two criteria for design, Sliding and shear failure and Settlement.

Different failure modes for foundation on rock
a) Cracking b) crushing c) Wedgeing
d) Puching e) shearing



Allowable load will depend on many factors

Occurrences During Excavation

- Undulating rock surface below a level ground
- Heterogeneity of rock mass (the bearing capacity may vary up to 10 times in apparently the same rock mass because of presence of localized fractures/shear zones/clay gauge/clay
- weathering/alternate hard land soft beds, etc.
- Solution and gas cavities;
- Wetting, swelling and softening of shales/ phyllite & expansive clays,
- Bottom heave;
- Potential unstable conditions of the slope
- High in situ horizontal stresses.

b) Adjacent Construction activities

- Blasting (Controlled blasting techniques such as line drilling, cushion blasting and pre-splitting are available if it is necessary to protect the integrity of the work just outside the excavation);
- Excavation
- Ground water lowering (except in highly pervious sedimentary rock, this phenomenon is rare in most of igneous and metamorphic rocks)

c) Other Effects

- Scour and erosion (in case of abutments and piers);
- Frost action
- Flooding (only erodible rocks like sale and phyllite) and
- Undesirable seismic response of the foundation.

Net allowable bearing pressure (q_a) based on rock material (IS:12070 - 1987)

Material	q_a (MPa)
Massive crystalline bedrock including granite, diorite, gneiss, trap, hard limestone and dolomite	10.0
Foliated rocks such as schist or slate in sound condition	4.0
Bedded limestone in sound condition	4.0
Sedimentary rock, including hard shales and sandstones	2.5
Soft or broken bedrock (excluding shale) and soft limestone	1.0
Soft shale	0.4

Net allowable bearing pressure (q_a) as per RMR (IS: 12070 - 1987)

Classification no.	*I*	*II*	*III*	*IV*	*V*
Description of rock	Very good	Good	Fair	Poor	Very poor
RMR	100–81	80–61	60–41	40–21	20–0
q_a (MPa)	6.0–4.5	4.5–2.9	2.9–1.5	1.5–0.6	0.6–0.4

IS 12070-1987 recommendations on rock foundations

- The permissible settlement for calculation of safe bearing pressure from plate load test should be taken as 12mm even for large loaded areas. The low value for settlement of foundation is due to heterogeneity of rocks.
- In case of rigid structures like silos, the permissible settlement may be increased judiciously, if required.
- Where site is covered partly by rocks and partly by talus deposits or soil, care should be taken to account for heterogeneity in deformability of soil and rocks.
- It is recommended that plate load tests be conducted on talus or soil and bearing pressure be recommended considering 12 mm settlement.

Estimation of bearing capacity

Tarzaghi's expression may be adopted assuming general shear failure. If the loaded area much smaller

$$q_{ult} = 1.2\, c'\, N_c + 0.5\, \gamma\, B\, N_r \qquad q_{ult} = \sigma_{ci}\, (RQD/100)^2$$

When no test data (c' & φ') is available

For heavily fractured rock

When rock mass is heavily fractured, and the foundation is located at some depth

$$q_{ult} = \gamma\, D_f \tan^4(45 + \phi'/2)$$

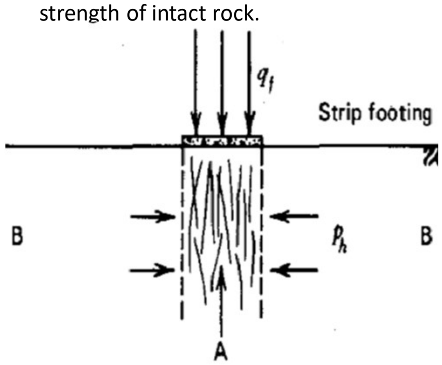
By considering the crushing of rock under the footing

$$q_{ult} = \sigma_{ci}\, [2/(1-\sin\phi')] \quad \text{or} \quad q_{ult} = \sigma_{ci}\, (N_\phi + 1)$$

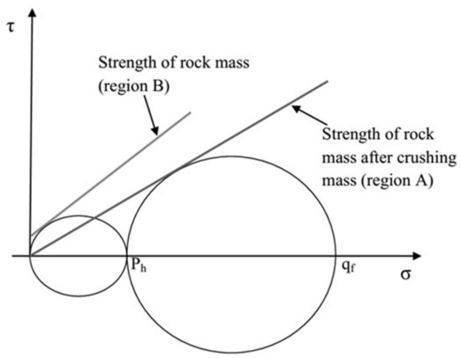
$$N_\phi = \tan^2(45 + \phi'/2),\ \phi' = \text{friction angle of intact rock}$$

Analysis of bearing capacity on rock (Goodman, 1989)

It is found that in a isotropic rock, safe bearing pressure often occurs at a settlement approximately equal to 4-6% of the footing width (Goodman, 1989).
Strength of the crushed rock under footing may be analysed considering zone A and B.
Considering the zone B in uniaxial compression and zone triaxial compression with confinement P_h , the maximum load q_f. The bearing capacity of rock is dependent on the residual strength of rock in the post peak region rather than the compressive strength of intact rock.



Footing resting on rock



Analysis of bearing pressure on rock

PRESSURE BULBS - IN ELASTIC HALF PLANE (Goodman, 1989)

The deformation behaviour of the near by rock mass for a rock foundation may be evaluated assuming elastic and isotropic material behaviour using following Eq.

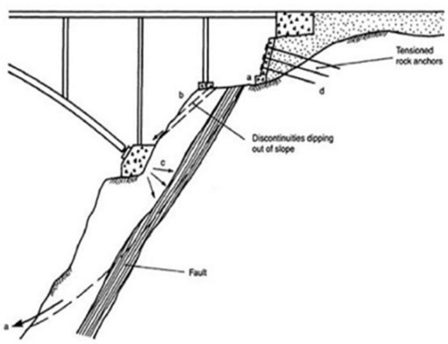
$$\text{displacement (u)} = \frac{C.P.(1-\nu^2)\, a}{E}$$

C= Constant dependent on the boundary condition. For perfectly rigid plate C= π/2 and for flexible plate, C=1.7.
P = The plate pressure (Contact force per unit plate area)
a = Plate radius
E, ν = Elastic modulus and Poisson's ratio respectively

DESIGN REQUIREMENTS - FOUNDATIONS

Retaining wall stabilized with reinforced concrete buttress & rock bolts

Bridge abutment founded on rocks



SETTLEMENT IN ROCKS (Schleicher, 1926)

$$\delta = \frac{[S_f \, P \, B \, (1-\mu^2)]}{E_j}$$

δ - settlement in the direction of loading

P - uniformly distributed load intensity

B - width or diameter of the loaded area

μ - Poisson's ratio of rock mass

E_j - deformation modulus

S_f - shape factor , reflecting shape of the loaded area and the location of the point where settlement is required.

For a given settlement of footing, the settlement of plate is obtained by using the following formulae

For massive or sound rock,

$$\frac{S_p}{S_f} = \frac{B_p}{B_f}$$

For laminated or poor rocks

$$\frac{S_p}{S_f} = \left[\frac{B_p}{B_f} \times \frac{(B_f + 30)}{(B_p + 30)}\right]^2$$

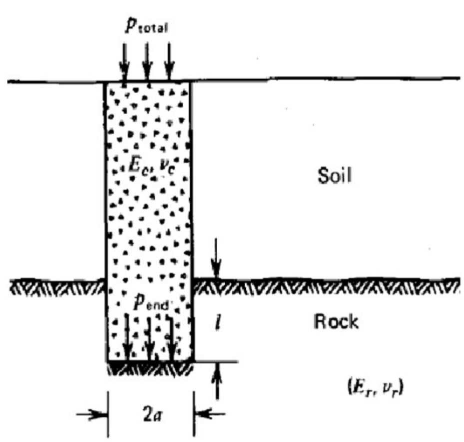
Where,

S_p = Settlement of plate (mm)

S_f = Settlement of footing (mm)

B_f = Size of Footing (cm)
B_p = Size of Plate (cm)

A Rock socketed pier



ROCK SOCKETED PIERS

The deformation at the base of the piers may be determined using Poulos and Davis (1968)

$$\omega_{base} = \frac{\left(\frac{\pi}{2}\right) p_{end}(1 - v_r^2)\, a}{E_r\, n}$$

p_{end} = normal pressure at the lower end of the pier or pile
v_r and E_r =Poisson's ratio and elastic modulus of the rock
a = radius of the lower end of the pile or pier.
n = factor depending on the relative depth and on v_r

Effect of embedment depth 'l' on displacement of a rigid plate

l/a	0	2	4	6	8	14
n: $\nu_r = 0$	1	1.4	2.1	2.2	2.3	2.4
n: $\nu_r = 0.3$	1	1.6	1.8	1.8	1.9	2.0
n: $\nu_r = 0.5$	1	1.4	1.6	1.6	1.7	1.8

- The end bearing load (P_p) of a pile/pier is given by

$$P_p = \pi D^2 \frac{q_{ult}}{4}$$

- D = Diameter or width of pile/ pie
- q_{ult} = ultimate bearing capacity of rock
- The load carried by the shaft surface shear resistance (P_s) is given by,

$$P_s = \pi D \; L \, \tau_s$$

- L = length of the pile/pier
- τ_s = shear strength of rock on the surface of the shaft

Therefore, the ultimate load carrying capacity (P) of pile/pier is given by

$P = P_p + P_s$

Maximum shear strength developed on the shaft to be estimated from the uniaxial compressive strength of the intact rock as follows

$$\tau_s = 0.3\ \sigma_{ci} \text{ or } 0.5\ \sigma_{ci}^{0.5}$$

adopting a suitable factor of safety (usually 3), the safe load carrying capacity of pile/pier is given by

$$P_{safe} = \frac{P_P + P_s}{F}$$

Shear resistance on piles socketed into rock from various researchers

Author	τ_s
Rosenberg and Journaux (1976)	$0.4\sigma_{ci}^{0.4}$
Poulos and Davis (1980)	$0.15\sigma_{ci}$
Standard DIN 4014 (1980)	0.08 for σ_{ci} = 0.5 MPa 0.5 for σ_{ci} = 5 MPa 0.5 for σ_{ci} = 20 MPa
Williams et al. (1980)	$0.15\sigma_{ci}$
Hovarth et al. (1983)	$(0.2–0.3)\sigma_{ci}^{0.5}$
Canadian Geotech Manual (1985)	$(0.2–0.33)\sigma_{ci}^{0.5}$
Rowe and Armitage (1987)	$(0.45–0.6)\sigma_{ci}^{0.5}$
Wyllie (1992)	$(0.4–0.6)\sigma_{ci}^{0.5}$
Fleming et al. (1992)	$0.4\sigma_{ci}^{0.5}$ for $\sigma_{ci} < 0.5$ MPa
Kulhawy and Phoon (1993)	$(0.22–0.67)\sigma_{ci}^{0.5}$
Hooley and Lefroy (1993)	$0.3\sigma_{ci}$
Carubba (1997)	$(0.13–0.25)\sigma_{ci}^{0.5}$
Zhang and Einstein (1998)	$(0.4–0.8)\sigma_{ci}^{0.5}$

HOEK–BROWN STRENGTH CRITERION FOR FRACTURED ROCK MASSES

- Strength of fractured rock masses, an empirical method (Hoek and Brown) in which the shear strength is represented as a curved Mohr envelope.
- This strength criterion was derived from the Griffith crack theory of fracture in brittle rock as well as observations of the behaviour of rock masses in the laboratory and the field
- Hoek and Brown introduced their failure criterion to provide input data for the analyses required for the design of underground excavations in hard rock. The criterion started from the properties of intact rock, and then introduced factors to reduce these properties based on the characteristics of joints in a rock mass

- lack of suitable alternatives, the criterion was soon adopted by the rock mechanics community and its use quickly spread beyond the original limits used in deriving the strength reduction relationships.
- enhancements were the introduction of the idea of ‘undisturbed’ and ‘disturbed’ rock masses and of a modified criterion to force the rock mass tensile strength to zero for very poor quality rock masses

The original Hoek–Brown strength criterion was defined in terms of the principal stress

$$\sigma_1' = \sigma_3' + \sigma_{ci}' \left(m \cdot \frac{\sigma_3'}{\sigma_{ci}} + s \right)^{0.5}$$

m and s are material constants; s = 1 for intact rock.

Hoek (1990) discussed the derivation of equivalent friction angles and cohesive strengths for various practical situations. These derivations were based upon tangents to the Mohr envelope derived by Bray. Hoek (1994) suggested that the cohesive strength determined by fitting a tangent to the curvilinear Mohr envelope is an upper bound value and may give optimistic results in stability calculations. Consequently, an average value, determined by fitting a linear Mohr–Coulomb relationship by least squares methods, may be more appropriate. In the 1994 paper, Hoek also introduced the concept of the generalised Hoek–Brown criterion in which the shape of the principal stress plot or the Mohr envelope could be adjusted by means of a variable coefficient ain place of the 0.5 power term.

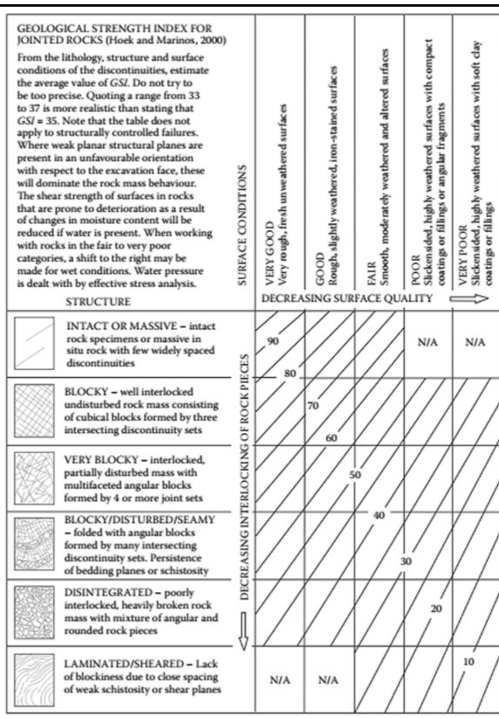
In addition to the changes in the equations, it was also recognised that the Rock Mass Rating of Bieniawski was no longer adequate as a vehicle for relating the failure criterion to geological observations in the field, particularly for very weak rock masses. This resulted in the introduction of the geological strength index (GSI) by Hoek, Wood and Shah (1992), Hoek (1994) and Hoek, Kaiser and Bawden (1995).

The GSI provides a system for estimating the reduction in rock mass strength for different geological conditions. Values of GSIare related to both the degree of fracturing and the condition of fracture surfaces

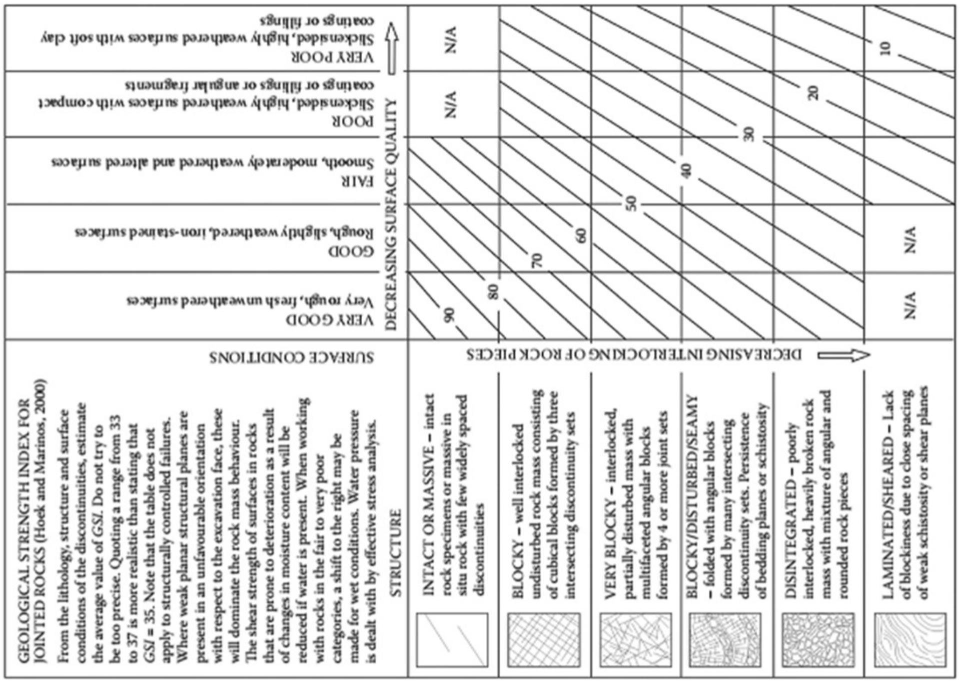
Generalised Hoek–Brown strength criterion

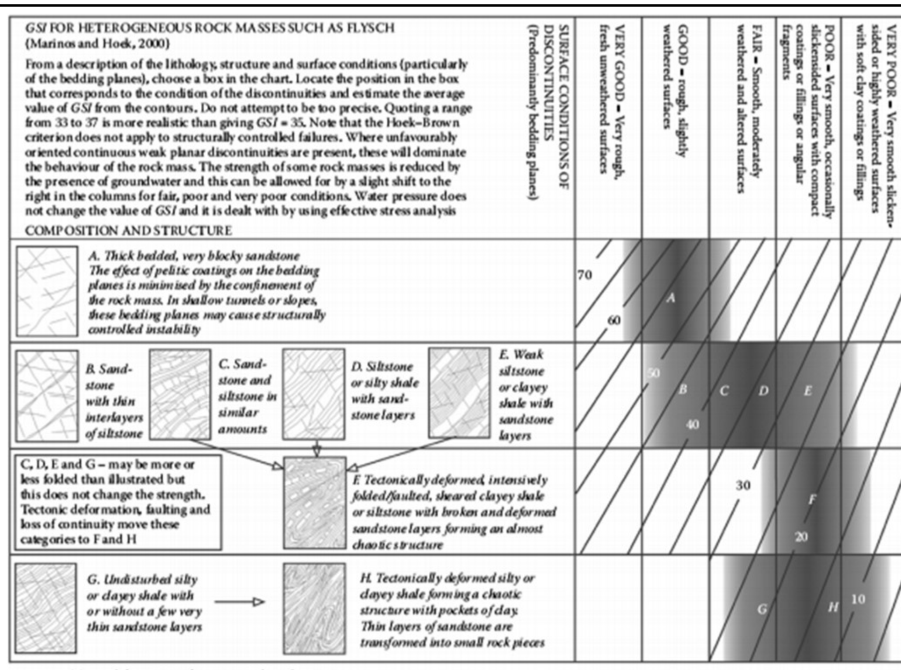
- An important limitation of the Hoek–Brown strength criterion is that it applies to isotropic rock masses, and it is not applicable to highly anisotropic masses such as interbedded shale and sandstone sequences, and strongly foliated metamorphic rock.

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \cdot \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

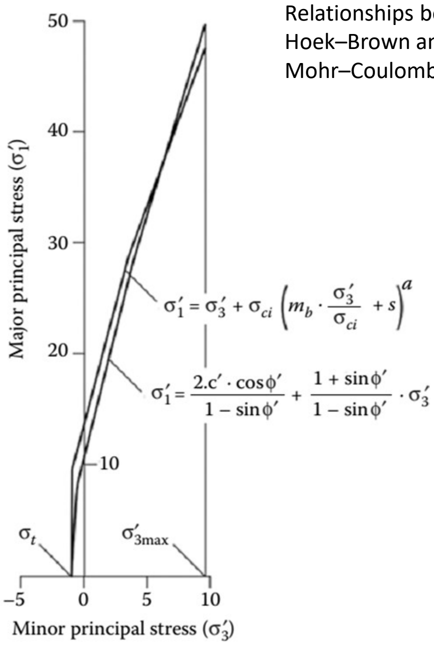


GSI values characterising blocky rock masses on the basis of particle interlocking and discontinuity condition





GSI values characterising heterogeneous rock masses on the basis of foliation and discontinuity condition.



Relationships between major and minor principal stresses for Hoek–Brown and equivalent Mohr–Coulomb criteria

$$m_b = m_i \cdot \exp\left(\frac{GSI - 100}{28 - 14D}\right)$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

Dis a factor that depends upon the extent of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses, to 1 for very disturbed rock masses

- UCS of Rock Mass

$$\sigma_c = \sigma_{ci} \cdot s^a$$

The tensile strength of a rock mass

$$\sigma_t = -\frac{s \cdot \sigma_{ci}}{m_b}$$

Modulus of deformation

$$E_m = \left(1 - \frac{D}{2}\right) \cdot \sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{(GSI-10)/40} \quad \text{(units: GPa)}$$

Table 5.3 Values of constant m_i for intact rock, by rock type (values in parenthesis are estimates)

			Texture			
Rock type	*Class*	*Group*	*Coarse*	*Medium*	*Fine*	*Very fine*
Sedimentary	Clastic		Conglomerates[a]	Sandstones 17 ± 4	Siltstones 7 ± 2	Claystones 4 ± 2
			Breccias[a]		Greywackes (18 ± 3)	Shales (6 ± 2)
						Maris (7 ± 2)
	Non-clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
Metamorphic	Non-foliated		Marble 9 ± 3	Hornfels (19 ± 4)	Quartzites 20 ± 3	
				Metasandstone (19 ± 3)		
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6	Gneiss 28 ± 5	
	Foliated[b]			Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
Igneous	Plutonic	Light	Granite 32 ± 3	Diorite 25 ± 5		
			Granodiorite (29 ± 3)			
		Dark	Gabbro 27 ± 3	Dolerite (16 ± 5)		
			Norite 20 ± 5			
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5)	Dacite (25 ± 3)	
				Andesite 25 ± 5	Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

Mohr–Coulomb criterion

The analysis of slope stability involves examination of the shear strength of the rock mass on the

sliding surface expressed by the Mohr–Coulomb

failure criterion. Therefore, it is necessary to

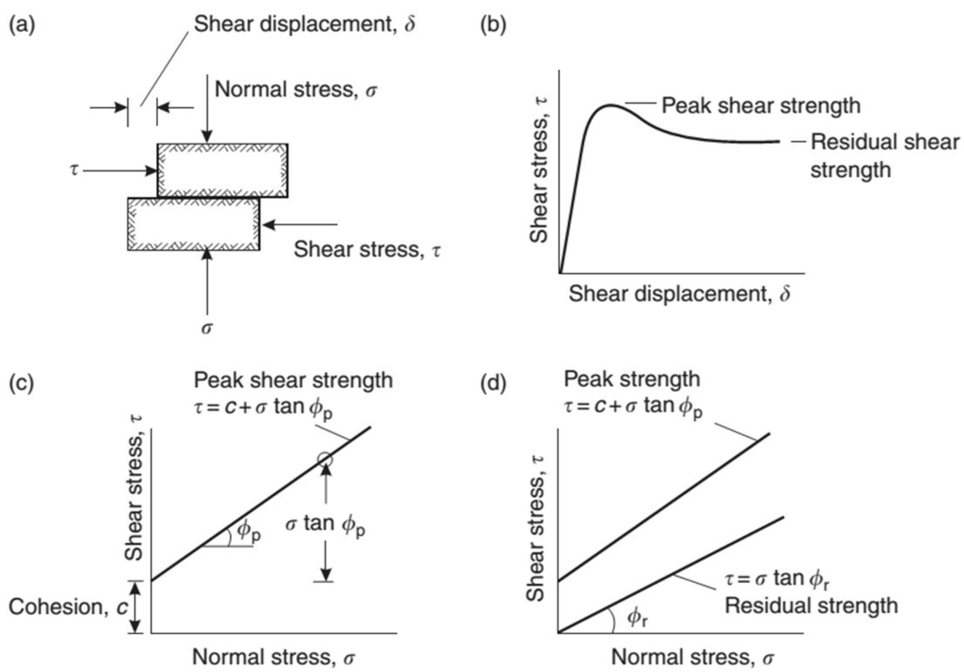
determine friction angles and cohesive strengths

that are equivalent between the Hoek–Brown

and Mohr–Coulomb criteria. These strengths are

required for each rock mass and stress range along

the sliding surface. This is done by fitting an average linear relationship to the curve generated by



Definition of shear strength of discontinuity surface; (a) shear test of discontinuity; (b) plot of shear displacement vs shear stress; (c) Mohr plot of peak strength; (d) Mohr plot of peak and residual strength.

At small displacements, the specimen behaves elastically and the shear stress increases linearly with displacement.

As the force resisting movement is overcome, the curve become non-linear and then reaches a maximum that represents the peak shear strength of the discontinuity.

Thereafter, the stress required to cause displacement decreases and eventually reaches a constant value termed the residual shear strength.

The cohesive component of the total shear strength is independent of the normal stress, but the frictional component increases with increasing normal stress.

For the residual strength condition, the cohesion is lost once displacement h as broken the cementing action.

Also, the residual friction angle is less than the peak friction angle because the shear displacement grinds the minor irregularities on the rock surface and produces a smoother, lower friction surface.

Typical ranges of friction angles for a variety of rock types

Rock class	*Friction angle range*	*Typical rock types*
Low friction	20–27°	Schists (high mica content), shale, marl
Medium friction	27–34°	Sandstone, siltstone, chalk, gneiss, slate
High friction	34–40°	Basalt, granite, limestone, conglomerate