Modeling of jointed rock

(1), (2)

o Strength as a function of joint orientation:

From before: $\tau_{max} = S_J + \sigma_d \tan \phi_J$

$$\sigma_d = \frac{1}{2}(\sigma_1 + \sigma_3) + \frac{1}{2}(\sigma_1 - \sigma_3) \cos 2\beta$$

$$\tau_d = \frac{1}{2}(\sigma_1 - \sigma_3) \sin 2\beta$$

Combining $\Rightarrow \frac{1}{2}(\sigma_1 - \sigma_3) \sin 2\beta = S_J + \left[\frac{1}{2}(\sigma_1 + \sigma_3) + \frac{1}{2}(\sigma_1 - \sigma_3) \cos 2\beta\right] \tan \phi_J$

after Algebraic ……………..

$$\frac{\sigma_1}{S_J} = \frac{2 + 2\left(\frac{\sigma_3}{S_J}\right)\tan \phi_J}{(1 - \cos\beta \tan \tau_J) \sin 2\beta} + \frac{\sigma_3}{S_J}$$

σ_1 & σ_3 : stresses at failure

- For what range of β, failure will be by sliding along joint??

* Conditions for sliding Along a Discontinuity:

Failure criteria for intact rock: $\tau_{max} = S_R + \sigma \tan \phi_R$

Failure criteria for discontinuous rock: $\tau_{max} = S_J + \sigma \tan \phi_J$ if $\sigma > \sigma_T$

Patton's law: $\tau_{max} = \sigma \tan (\phi_u + i)$ if $\sigma < \sigma_T$

$\Downarrow$

$$\text{If} \quad \left.\begin{aligned} \beta &> \frac{2\beta_1}{2} \\ &< \frac{2\beta_2}{2} \end{aligned}\right\} \text{ if sliding}$$

if not $\Rightarrow$ new crack

Failure will occur along the discontinuity of $2\beta_1 < 2\beta < 2\beta_2$
or $\beta_1 < \beta < \beta_2$.

alternatively $\psi_1 + \psi > \psi_2$

<u>Note</u>: The value of β_1 & β_2 are function of

β_1 & $\beta_2 = f(\sigma_1, \sigma_3, \phi_R, S_{R,} S_J, \phi_u, i, \sigma_T$

Failure is due to sliding along discontinuity

Failure through intact rock.

Course pack Fig. 6.6 pp. 53
6.7

H.W. # 7

Water Pressure affects in Discontinuous Rock:

$\sigma' + \sigma_T - P_w$

Pore water pressure required to cause failure must be considered separately for each of the three modes of failure.

For a particular ψ

a) Failure by riding over asperities

$$P_w = \sigma_3 + (\sigma_1 - \sigma_3)\left\{\sin^2 \psi - \frac{\sin \psi \cos \psi}{\tan(\phi_u + 2)}\right\}$$

b) Failure by shearing through asperities:

$$P_w = \frac{S_J}{\tan \phi_J} + \sigma_3 + (\sigma_1 - \sigma_3)\left\{\sin^2 \psi - \frac{\sin \psi \cos \psi}{\tan \phi_J}\right\}$$

c) Failure at intact rock:

$$P_w = \sigma_3 - \frac{\left[(\sigma_1 - \sigma_3) - 2\, S_R \tan(45 + \frac{\phi R}{2})\right]}{\tan^2(45 + \frac{\phi R}{2}) - 1}$$

Compute all three to find P_w minimum required to cause failure.

11. Deformability of rock masses (rather than intact rock) see Ch. 6 text.

11.1 Def.:

Modulus of permanent deformation.

M

11.2 In-situ Tests for determining rock mass modulus:

11.2.1 Plate Bearing test:-

i) Reaction Against Anchor

ii) Reaction Against an opposite wall

Theory: $$\varpi = \frac{G\,p\,(1-\nu^2)\,a}{E}$$

where ϖ = displacement measured (average of three readings)
p = plate pressure (applied)
a = plate radius
G = $\pi/2$ if plate is rigid or 1.7 if plate is flexible.

Typical Results:-

$$E = C\,a\,(1-\nu^2)\,\frac{P}{\overline{\varpi}_{\text{elastic component}}}$$

mass modulus

$$M = C\,a\,(1-\nu^2)\frac{P}{\varpi - \varpi_{\text{elastic}}}$$

If extensometers are used:

$$E = \frac{(1+\nu)\,P}{\varpi}\left[\frac{-Z^2}{\sqrt{Z^2+a^2}} + 2\,(1-\nu)\,\sqrt{Z^2+a^2} - (1-2\nu)\right]$$

ϖ = displacement at depth Z.

See fig. pp.58

11.2.2 Borehole Dilatometer:- pressure a borehole $\rightarrow$ Δu

See fig. pp. 59

$$E = (1-\nu)\Delta P\,\frac{a}{\Delta u}$$

easy test

11.3 Modulus of Fractured Rock (Rock mass) from intact Rock & Joint properties

A: Joint Testing

B: Intact Rock
stress-strain test

elastic compression of intact rock

$$= \frac{\sigma_n}{E_{rock}} \cdot S \qquad \text{(assuming S much larger than joint width)}$$

compression of joint $\delta = \dfrac{\sigma_n}{K_n}$

$$\text{Total Deformation} = \frac{\sigma_n}{E_{rock}} S + \frac{\sigma_n}{K_n}$$

$$\text{Strain in the rock mass} = \varepsilon_{rock\ mass} = \frac{\text{Total Deformatio n}}{S}$$

$$= \frac{\sigma_n}{E_{rock}} + \frac{\sigma_n}{K_n S}$$

$$E_{rock\ mass} = \frac{\sigma_n}{\varepsilon_{rock\ mass}} = \frac{\sigma_n}{\dfrac{\sigma_n}{E_{rock}} + \dfrac{\sigma_n}{K_n S}} = \frac{E_{rock} \cdot K_n S}{E_{rock} + K_n S}$$

Derivation for $G_{rock\ mass}$ follows in the same manner.

$$G_{rock\ mass} = \frac{G_{rock} \cdot K_s \cdot S}{G_{rock} + K_s \cdot S}$$

$\rightarrow$ Do both tests, to to field to measure S

$\rightarrow$ Do test on rock, go to field & do tests with small $\Rightarrow$ Extrapolate for larger S.

11.4 Rock Indices for estimating $E_{rock\ mass}$: (approximate)

a. Bieniawski (1978): E (GPa) = 2 (RMR) – 100 for RMR 55

b. using Modulus Reduction Factor $= \dfrac{E_{rock\ mass}}{E_{rock}} = f\ (RQD)$

see Fig. 7.2 from US army corp of engg.

H.W. # 8

Ch. #4 & 9

12. Slope Stability in Rocks

More complicated than soil — predetermine failure planes / it can fail in tension

12.1 Types of Failure

i. plane sliding
(one joint of orientation is significant)

ii. wedge-type failure
(Two joints sets (or more) are significant

iii) Toppling failure

12.2 Plane Sliding:

a) Kinematics

if $\delta > a$	no failure
if $\delta < a$	failure

For failure to be kinematically feasible $\delta < \alpha$

(the discontinuity must (daylight))

o stereographic plot presentation for (a)

represents dip vector δ_1

if $\delta = \delta_1$ failure is kinematically possible

if $\delta = \delta_2$ failure is not possible

it is enough for δ to be > α failure to failure, but δ have to be > δ_{min}.

b) strength : (ϕ):

since $\phi > \alpha$: block will <u>not</u> slide.

$\therefore$ $\phi < \alpha$ block will slike.

o stereonet presentation for (b)

If dip rector plots in this area, failure
can't occur.

$\therefore$ joint is not steep enough

stereonet for (a) & (b).

Combining the 2 criteria:

failure can only <u>occur</u> if dip
vector plots in shaded area.

joint must both daylight and its angle must exceed ϕ_J.

Note: This only due to self weight of block.

- still wt. of adjacent blocks.
- still wt. of adjacent water pressure.

Example of using stereonet for vector addition:

Ex.1 given two forces: 200 = acting in the direction N80^{o}W plunging 50^o
600 = acting in the direction S40^{o}W plunging 20^o

Find the resultant force and its direction.

Solution: (Procedure)

1. Find the common plane of the two forces.

rotate until they plot on same circle. → angle = 55^o

2. Determine the resultant force by parallelogram theorem. (draw to scale). Find resultant. Find angle = 13^o.

3. Plot direction of the resultant on stereonet.

S50^{o}W, 290^0

Forces acting on rock blocks:-

1. Block self weight.
2. Forces transmitted from adjacent blocks.
3. Forces due to water pressure.
4. Dynamic loads.
5. Reinforcement (rock bolts).

if $\phi < \phi_J$ stable

How to plot ϕ-circle?

To plot the friction circle on stereonet:

1. Locate the normal to the failure plane, $\hat{n}$
2. Plot two points on the diameter of the circle each being ϕ_J^o from the normal (measured along the diameter).
3. Construct ϕ-circle having the line between the two points (in step # 2) as the diameter.

If resultant vector, $\hat{r}$ is inside the circle, failure doesn't occur.

factor of safety against sliding

$$\text{F.S.} = \frac{\tan \phi_{available}}{\tan \phi_{required}}$$

Ex. 1. A joint strikes S30^{o}E and dips 60^o NE

ϕ_J 25^o

resultant of all forces $\hat{r}$ acts in the direction S50^{o}W and plunges 20^o,

Find the factor of safety.

Procedure

1. Plot the friction circle for the joint.

2. Plot resultant force $\hat{r}$.

3. Find angle between $\hat{r}$ and $\hat{n}$. (put them in same greater circle)

   $\Rightarrow \quad \phi_{reqd.} = 15^o$

4. $\therefore \; \phi_{available} = \phi_J = 25^o$

   $$\therefore \; \text{F.S.} = \frac{\tan 25}{\tan 15} = 1.74$$

Ex. 2 Assume that the only force acting on the block is its self-weight. Determine the minimum bolt force and direction required to raise the F.S. to 1.0 for a block weighing 20 tons.

i.e. dipping 90^o $\Rightarrow$ plot @ the center of streonet.

$\hat{w}$ = weight vector

$\rightarrow$ $\hat{w}$ not inside friction circle $\Rightarrow$ F.S. << 1.0.

Procedure:

1) $\hat{w}$ will plot @ center of streonet $\Rightarrow$ F.S. << 1.0.

2) Find the point on the ϕ-circle at which $\hat{r}$ will make the smallest angle with $\hat{w}$.

answer. 35^o

if bolt driven S60^oW

3) Construct force diagram

own wt.
angle of resultant } bolt force & resultant

4. Determine a line of minimum length from tip of $\hat{w}$ to the $\hat{r}$ line.

$|B| = 11$ Tons 35^o up from horizontal.

<u>Ex. 3.</u> What bolt force would be required to achieve F.S. = 2.5.

<u>Sol.</u> $$\text{F.S.} = \frac{\tan \phi_{available}}{\tan \phi_{reqd.}} = \frac{\tan 25°}{\tan \phi_{reqd.}} = 2.5$$

$$\tan \phi_{reqd.} = \frac{\tan 25°}{2.5} = \qquad \Rightarrow \phi_{reed.} = 10.5^o$$

min. angle between $\hat{w}$ and $\phi_{circle} = 49.5^o$

$|B| = 15$ Tons 49.5^o up from horizontal

Ex. 4. What both force would be required if the bolt are to be driven due west.

$|B| = 16.5$ T

Analysis of Plane Slides by traditional Block Sliding Analysis

Ref. Book (Hock & Bray 1977)

#1.

Crack intercepts crest of slope

$$W = \frac{1}{2} \gamma H^2 \left[\left(1 - (\tfrac{Z}{H})^2 \right) \cot \delta - \cot \alpha \right]$$

#2.

Crack intercepts face of the slope.

$$W = \frac{1}{2} \gamma H^2 \left[(1 - \tfrac{Z}{H})^2 \cot \delta (\cot \delta \tan \alpha - 1) \right]$$

Area of sliding surface: $A = \dfrac{H - Z}{\sin \delta} * 1$

Resultant of water pressure along the vertical crack.

$$V = \int_o^{Z_w} Z \, dZ = \frac{1}{2} \gamma Z_w^2$$

Resultant of water pressure acting on the sliding surface = $U = \gamma_w \frac{Z_w}{2} \cdot A \quad = \frac{1}{2} \gamma_w Z_w A.$

Shear Forces Along Sliding Surface:

Forces resisting shear:

$(S_J + \sigma_n' \tan \phi_J)\ A$

$$\sigma_n' = \sigma - u = \frac{W \cos \delta - V \sin \delta - U}{A}$$

Sliding occurs when the driving shear forces = forces resisting shear.

$$\text{F.S.}\ (W \sin \delta + V \cos \delta) = S_J\ A + (W \cos \delta - V \sin \delta - U) \tan \phi_J$$

For Case #1:

$$\text{F.S.}\left[\frac{1}{2}\gamma H^2\ \left[\left(1-\left(\tfrac{Z}{H}\right)^2\right)\cot\delta - \cot\alpha\right]\sin \delta + \tfrac{1}{2}\gamma_w Z_w^2 \cos \delta\right]$$

$$= S_J\ \tfrac{H-Z}{\sin\delta} + \left[\frac{1}{2}\delta H^2\ \left[\left(1-\left(\tfrac{Z}{H}\right)^2\right)\cot \delta - \cot\alpha\right]\cos \delta\right.$$

$$\left. - \frac{1}{2}\gamma_w Z_w^2 \sin \delta - \frac{1}{2}\gamma_w Z_w A\right] \tan \phi_J$$

Since we have some control over α_1, we can solve for it.

$$\text{Cot } \alpha = \frac{[a\,(F.S. \sin\delta - \cos\delta \tan\phi) + U \tan\phi + V\,(\sin\delta \tan\phi + F.S. \cos\delta)] - S_J A}{b\,(F.S. \sin\delta - \cos\delta \tan\phi)}$$

where $a = \frac{1}{2}\gamma H^2 \left(1 - \left(\frac{Z}{H}\right)^2\right) \text{Cot } \delta$

$b = \frac{1}{2}\gamma H^2$

ΔS_J is more important for steep slopes (α is large)

$\Delta \phi_J$ is more important for high slopes (H is large)

Drainage of water can be very effective in stabilizing rock slopes.

* Wedge Failure Analysis

a) <u>Kinematics:</u> for sliding of a wedge, use $\hat{I}$ (the interaction of 2 planes in place of the dip vector $\hat{D}$.

$\therefore$ if the plunge of $\hat{I}$ is less than α, the wedge sliding is kinematically possible.

- Find normals to two planes $\hat{n}_1$, $\hat{n}_2$
- Rotate until $\hat{n}_1$ & $\hat{n}_2$ on one greater circle.

Failure can occur in one of three ways.

1. if $\hat{D}_1$ daylights
2. if $\hat{D}_2$ daylights plane sliding
3. if $\hat{I}_{12}$ daylights $\rightarrow$ sliding

If we have 3 sets of joints; Failure can occur on:

1. $\hat{D}_1$ daylight
2. $\hat{D}_2$ daylight
3. $\hat{D}_3$ daylight
4. $\hat{I}_{12}$
5. $\hat{I}_{13}$
6. $\hat{I}_{23}$

b) <u>Strength</u>: If ϕ_J > plunge of $\hat{I}$ failure cannot occur.
Furthermore, for very acute (steep) wedges, considerable strength is obtained from roughness along the discontinuities, so $\hat{I}$ can often be steeper than ϕ_J without failure.

Conventional Wedge Analysis

Ref. Hock & Bray "Rock Slope Engineering"
Institution of Mining & Metallurgy 1981

Geometry:

ψ_{fi} = inclination of slope, slope measured in the view at right angles to the line of intersection.

ϕ_J = joint friction angle.

actual slope is > ψ_{f2}

ψ_I = plunge of line of intersection.

Sliding will occur if $\psi_{fi} > \psi_I > \phi$

Forces:

Assume sliding is resisted by friction only.

$$\text{F.S.} = \frac{(R_A + R_B)\tan\phi_J}{W\sin\psi_i}$$

$$\Sigma F_H \Rightarrow \quad R_A \sin\left(\beta - \tfrac{1}{2}\xi\right) = R_B \sin\left(\beta + \tfrac{1}{2}\xi\right)$$

ΣF_r along line of intersection $\Rightarrow R_A \cos\left(\beta - \tfrac{1}{2}\xi\right) - R_B \cos\left(\beta + \tfrac{1}{2}\xi\right) = W \cos\psi_i$

Solving for R_A & R_B and adding

$$R_A + R_B = \frac{W \cos \psi_i \sin \beta}{\sin \frac{1}{2} \xi}$$

$$\therefore \quad F.S. = \frac{\sin \beta}{\sin \frac{1}{2} \xi} \cdot \frac{\tan \phi_J}{\tan \psi_i}$$

f.s. for plane sliding

$(F.S.)_{wedge} = K * (F.S.)_{plane}$

<u>Fig. 96:</u>

ξ : angle between joints

β : angle of tilt of line of intersection

<u>Note</u>: This is limited to joints having their normals coplan
(same plan) 180^o in phase.

$F.S. = A \tan \phi_A + B \tan \phi_B$

A & B see pp.211 - end.

<u>Ex</u>.	Dip	Dip. direction	ϕ_J
Plane A	40^o	165	35^o
Plane B	70^o	285	20
	-----	-----	
Difference	30^o	120	

A = 1.5 , B = .7

F.S. = 1.5 tan 35 + 0.7 tan 20 = 1.3

This method is good better : ξ & β is difficult to find in field.

If F.S. $\geq$ 2 $\therefore$ Wedge failure is almost impossible.

o Wedge Analysis on stereographic projection.

Text pp.---- Fig. 8.16

1. Find $\hat{n}_1$ and, $\hat{n}_2$ then $\hat{I}_{12}$
2. Draw greats circle through $\hat{n}_1$, I_{12} and $\hat{n}_2$, I_{12}
3. Construct ϕ circles for n_1 and n_2.
   This gives the four points of intersection: p, q, s & t.
4. Construct great circles connecting p.s. and q.t.

if resultant of all forces.

Toppling Failure

a) flexural toppling

b) block toppling

c) combination of flexural and toppling

- Static Analysis of Block Toppling

a. Single block:

if W is to left of $\Rightarrow$ Failure

condition if impending failure

$$\text{Cot } \alpha = \frac{\Delta y}{\Delta x}$$

$$\text{Cot } \alpha < \frac{\Delta y}{\Delta x} \Rightarrow \text{failure will occur.}$$

Long narrow blocks on steep slopes are more susceptible to failure.

b. Multiple blocks:

Resisting Moments:

$$P_{n-1}\ (L_n) + \mu\ P_n\ \Delta x + W \text{ Cos } \alpha \frac{\Delta x}{2}$$

Overturning Moments

$$P_n\ (M_n) + \sin\ \alpha \frac{\Delta y}{2}$$

Setting R.M. = O.M. , toppling will occur if

$$P_{n-1} < \frac{P_n\ (M_n - \mu \Delta x) + \frac{W}{2}\ (\Delta y \sin\ \alpha - \Delta x \cos\ \alpha)}{L_n}$$

Many equations

Ref. Zanbak, Caner "Design Charts for Rock Slopes Susceptible to Toppling" J. of Geot. Engg., ASCE, Vol. 109, No. 8.

Stereographic Analysis for Flexural Toppling:

major principle direction
$\therefore$ no stress normal to slope

Topping will occur only if there is relative motion between dipping layer.

slippage must occur

Criterion for toppling failure: resultant should be outside ϕ-circle.

If we lay off an angle ϕ_J from the normal to the dipping beds, and it falls outside of the slope, then failure will occur because the direction of applied compression is outside of the ϕ-circle.

Toppling will occur if $(90-\delta) + \phi_J < \alpha$

Or $\delta > 90 + \phi_J - \alpha$

ϕ_J

ϕ_J

Toppling No Toppling

If $\hat{n}$ in this region, flexural toppling will occur if strike is

if > 30^o $\therefore$ toppling

is not towards excavation.

H.W. #8

Ch. 8 Prob. # 1 : Use: Set 1 Strike N40^oE

Prob. # 2: Use $\phi_J = 30^o$

Prob. # 3: Use: dip 55^oNE

P = 600 Tons

$\phi = 35^o$

13.0 Foundation on Rock

- Excessive settlement → compressibility of joints
- Bearing capacity

Fig. 9.1

Karstic: Ca O_2, dissolvable sinkable in Florida

a) Shallow Foundations:

spread footings

b) Deep Foundations

H. piles
Precast conc. — Piles
Pipe piles

Pier
Socketed
into rock

13.1 Shallow Foundations:

For intact rock in its elastic range:

Settlement may be predicted by:

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

where

C = - $\pi/2$ if rigid
- 1.7 if flexible

P : applied stress

E : Young's Modulus

ν : Poisson's ratio

a : radius of footing (or equivalent)

o If rock is not homogeneous …… se same as soil

Stress distribution beneath footings.

a) for homogeneous isotropic rock, elasticity solution are generally available. $\rightarrow$ point load …… integrate to get it for any shape.

b) for heterogeneous, anisotropic rock, finite element methods may be required.

c) Fig. 9.7 Line load

Vertical

Horizontal

if there is a joint then tension
will in opening crack ⇒ stress will not be
transmitted if total stress is tension.

Inclined

Fig. 9.8 not intact rock

resultant can't go
outside ϕ_J

⇒ slippage ⇒ realignment of stresses

Close Form Solution

1. Resolve Q & P

// & ⊥ to pidding planes

1977 Bray

$$\sigma_r = \frac{h}{\pi r}\left[\frac{X\cos\beta + yg\sin\beta}{(\text{Cot}\,\beta - g\sin^2\beta)^2 + h^2\sin^2\beta\,\text{Cos}^2\beta}\right]$$

where $g = \left(1 + \frac{E}{(1-\nu^2)k_n S}\right)^{1/2}$

$$h = \left\{\left(\frac{E}{1-\nu^2}\right)\left[\frac{2(1+\nu)}{E} + \frac{1}{k_s \cdot S}\right] + 2\left(g - \frac{\nu}{1-\nu}\right)\right\}^{1/2}$$

E, ν : are intact rock properties

k_n , k_s : are joint stiffnesses

S = Spacing

Fig. 9.9

Fig. 9.10

-----

* Strains and settlements beneath loaded rock masses.

<u>Procedure</u>

1. determine the stress distribution.

2. determine the equivalent elastic properties

$$G_{ns} = G_{nt} = \frac{G_{Rock} K_s S}{G_{Rock} + K_s S}$$

Similarly

$$E_n = \frac{E_{Rock} K_n S}{E_{Rock} + K_n S}$$

$$E_s = E_t = E_{rock}$$

$$\nu_{sn} = \nu_{tn} = \frac{\varepsilon_n}{\varepsilon_s} = \nu_{rock}$$

by symmetry,

$$\frac{\nu_{sn}}{E_s} = \frac{\nu_{ns}}{E_n} \qquad \therefore \quad \nu_{ns} = \nu_{nt} = \frac{E_n}{E_s} \nu_{sn}$$

$$= \frac{E_{rock} K_n S}{E_{rock} + K_n S} \cdot \frac{\nu_{rock}}{E_s = E_{rock}}$$

$$= \frac{K_n S \cdot \nu_{rock}}{E_{rock} + K_n S}$$

3. Employ the constitutive relationship for transversely isotropic media:

$$\begin{Bmatrix} \varepsilon_n \\ \varepsilon_s \\ \varepsilon_t \\ \gamma_{ns} \\ \gamma_{nt} \\ \varepsilon_{st} \end{Bmatrix} = \begin{bmatrix} \frac{1}{E_n} & \frac{-\nu_{sn}}{E_s} & \frac{-\nu_{sn}}{E_s} & 0 & 0 & 0 \\ \frac{-\nu_{sn}}{E_s} & \frac{1}{E_s} & \frac{-\nu_{st}}{E_s} & 0 & 0 & 0 \\ \frac{-\nu_{sn}}{E_s} & \frac{-\nu_{st}}{E_s} & \frac{1}{E_s} & 0 & 0 & 0 \\ 0 & 0 & 0 & & 0 & 0 \\ 0 & 0 & 0 & 0 & & 0 \\ 0 & 0 & 0 & 0 & 0 & \end{bmatrix} \begin{Bmatrix} \sigma_n \\ \sigma_s \\ \sigma_t \\ \tau_{ns} \\ \tau_{nt} \\ \tau_{st} \end{Bmatrix}$$

Bearing Capacity of Shallow Foundations on Rock.

Fig. 9.5

∴ The largest horizontal stress that can be developed to support the rock beneath the footing is the unconfined compressive strength, q_u.

since $\sigma_{1f} = \sigma_{3f} \tan^2 (45 + \phi/2) + q_u$

∴ $q_f = q_u \tan^2 (45 + \phi/2) + q_u$

$q_f = q_u (N_\phi + 1)$

bearing capacity factor ⇒ $N_\phi = \tan^2 \left(45 + \frac{\phi}{2}\right)$

$q_{all} = \frac{q_f}{F.S.} = \frac{q_f}{3}$ especially F.S. = 3.0

o Table 9.2: most rock of the region is very conservative

New York Detroit → largest values → highest buildings.

Drop Foundations on Rock:

a) bearing capacity increases with depth due to increase in confinement.

$\sigma_v \neq 0 \quad \Rightarrow$

b) bearing capacity doesn't increase with depth.

Settlement of Deep Foundations:
- rock
- pier: pile or corrosion

a) due to rock deflection for

elastic material : $$w_{base} = \frac{\frac{\pi}{2} P_{end} (1 - \nu^2)\, a}{E \cdot n}$$

n = f (embedment depth, radius and ν)

ℓ/a \ ν	0	2	4	6	8	14
0	1.0	1.4	2.1	2.2	2.3	2.4
.3	1.0	1.6	1.8	1.8	1.9	2.0
.5	1.0	1.4	1.6	1.6	1.7	1.8

b) due to concrete deformation:

$$\omega_{concrete} = \frac{P\ \ell_c + \ell}{E_c}$$

But P is not constant along the pile.

c) Correction for load transfer due to side friction:-
Neglect side friction through the soil.

derived in Appendix : same as one for consolidation

$$\sigma_Z = P_{max} \cdot e - \left[\frac{2\ \nu_c\ \tan\ \phi_{rc}}{1 - \nu_c + (1 + \nu_r)\ \frac{E_c}{E_r}} \cdot \frac{Z}{a} \right]$$

ν_c , ν_r : Poisson's ratio of concrete & rock

ϕ_{cv} : friction angle for rock/concrete & interface

E_c , E_v : Young's moduli for concrete & rock.

$$\therefore\ \Delta w = \frac{1}{E_c} \int_o^{\ell} (P_{max} - \sigma_z)\ dZ$$

d) $\therefore$ Total Deformation (settlement) = $w_{base} + w_{concrete} - \Delta w$

Determining depth of "socketing" into rock to insure against bearing capacity failure.

(1) Ref: Osterberg & Gill
F.E.M. study (1971)

found that even small embedment (socketing) into rock greatly reduces P_{end} by taking load in side friction.

Therefore shaft diameters can be reduced.

Fig. 9.18

(2) Ladanyi (1977) proposed a procedure for determining socketing depth.

o Assumptions:

a) no load transfer along concrete/soil interface.

b) bond between concrete and rock broken
$\Rightarrow$ use residual strength.

$$\sigma_{max} = \frac{P}{\pi a^2}$$

$$\sigma_z = \sigma_{max}\, e$$

For soft rock:

$\tau_{residual} = \alpha\, S_u$

S_u : undrained shear strength

α : reduction factor, $0.3 < \alpha < 0.9$, typically $\alpha = 0.5$

For hard rock: $\tau_{res.} \simeq \frac{q_u}{20}$

where q_u = unconfined comp. strength.

<u>Procedure</u>:

1. Determine shaft diameter "a" based on concrete strength.

2. Assume that $Q_p = 0$, then $\ell = \ell_1 = \frac{P}{2\pi a\ \tau_{all}}$

3. Choose $\ell = \ell_2$ such that $\ell_2 < \ell_1$.

4. Compute σ_Z @ $Z = \ell_2$.

5. If $\sigma_Z)_{@\ Z = \ell 2} > q_{all}$, then assume a new ℓ_2.

6. If $\sigma_Z\ )_{@\ Z = \ell 2} \leq q_{all}$, determine τ along perimeter of pile

$$\tau = \frac{1}{2\ \pi a\, \ell_2}\left[\left(1 - \frac{\sigma_Z)_{@\ \ell_2}}{\sigma_{max}}\right)P\right]$$

7. Repeat the procedure to find the shortest "ℓ" for which
   $\sigma)_{@\ \ell} \leq q_{all}$ and $\tau \leq \tau_{all}$

q_{all} ??? $\therefore$ for shallow foundation. No vertical stress

$$q_{all} = \frac{q_u\ (N_\phi + 1)}{F.S.}$$

For Deep Foundation

Since q_{all} as determined for shallow foundations would be very conservative, $\therefore$ use a smaller factor of safety, say 1.5 to 2.

Final : open book & notes.

(1) Geological Rock =
texture

(2) Permeability

(3) Testing - Brazilian

- 2 point load

(4) Triaxial

$\sigma_3 =$

$\sigma_1 =$ $\qquad \frac{\Delta V}{V} = \varepsilon_1 = \varepsilon_2 + \varepsilon_3$

ΔL_1

ϕ V & $\varepsilon_2 = \varepsilon_3 =$ ___

L_o

E $\qquad \varepsilon_1 = f(\sigma_1\ \sigma_2\ \sigma_3\ E, Y)$

$\varepsilon_2 =$

$E, V \rightarrow G$

$$G = \frac{E}{2(1+\nu)}$$

(5) Denatoric - stress
- strain

- <u>Disc.</u> Faults → shears → joints → fissures → microfissures
- Geo. class → Geometric → Igneous →
  - extrusive (volcanic)
  - intrusive (plutonic)

  Sedimentary - detrital - chemical
  - elastic - nonelastic

  Lithification: unconsolidated → consolidated
  - → cementation
  - → compaction
  - → dessication
  - → crystallization

  Metamorphic
  - → heat
  - → pressure
  - → chemically active fluds

  foliated unfoliated

  shale ↓ slate ↓ phyllite ↓ schist ↓ gneiss

- $n = \frac{V_P}{V_t}$
- $G_{s,r} \sum_{i=1}^{n} G_{s,i} \ V_i$
- $\gamma_{dry} = G_{s,r} \, \gamma_w \, (1-n)$
- $k = K.F. = K \frac{\gamma_w}{\mu}$ $q_i = k_i \frac{dh}{dx_i} A_L$

$$k = \frac{q \cdot \ell n \, (R_2 / R_1)}{dx_i}$$

$$k = \frac{\gamma_w}{6\mu} \left(\frac{e^2}{S} \right)$$

- Point load test
  - $d = 54$ mm
  - $L = 1.5\ D$
  - $I_s = P/D^2$
  - $Q_u = C\ I_s$

- $V_p^2 = E/\delta$

$$\frac{1}{V_\ell^*} = \sum_{i=1}^{n} \frac{C_i}{V\ell_i} \Rightarrow I_Q \,\% = \frac{V_\ell}{V_\ell^*} * 100\% \quad \text{no fissure} \quad I_Q = 100 - 1.6\, n\ \%$$

- $\text{RQD} = \dfrac{\sum LX''}{\sum L}$
- Failure $\rightarrow$ Tension
  $\rightarrow$ Shear
  $\rightarrow$ Compaction
- Unconfined comp. test $\frac{L}{D} = 2-3 \quad \Rightarrow \quad q_u = \dfrac{P_{max}}{A}$
- Triaxial Testing: strength = f (confining p.)
- Brazilian split cylinder test $\sigma_t = \dfrac{2P}{\pi dt}$
- Flex $\sigma_t = \dfrac{16}{3}\dfrac{PL}{\pi d^3}$ $\qquad I = \dfrac{\pi 6^4}{64}$
- Ring shear test $\tau_{max} = \dfrac{2P}{\pi d^2}$

* $\sigma_{ij} = \dfrac{1}{3}\sigma_{kk}\,\delta_{ij} + \tau_{ij}$

$$\left.\begin{array}{l} D_1 = \dfrac{\Delta\bar{\sigma}}{\Delta\bar{\varepsilon}} = 3K \quad \Rightarrow \\ \\ D_2 = \dfrac{\Delta\sigma_{1,\,dev}}{\Delta\varepsilon_{1,\,dev}} = 2G \quad \Rightarrow \end{array}\right\} \quad E = \frac{3D_1 D_2}{D_2 + 2D_1} \quad , \quad \nu = \frac{D_1 - D_2}{2D_1 + D_2}$$

$$G_{yx} = \frac{E_x}{2(1+\nu_{yx})} \quad \Rightarrow \quad G = \frac{E}{2(1+\nu)} \quad \Rightarrow \quad 2G = \frac{E}{1+\nu}$$

$$K = \frac{E}{3(1-2\nu)} \quad \Rightarrow \quad 3K = \frac{E}{(1-2\nu)}$$

$$\lambda = \frac{E\nu}{(1+\nu)(1-2\nu)}$$

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{Bmatrix} = \begin{bmatrix} \frac{1}{E} & \frac{-\nu}{E} & \frac{-\nu}{E} & 0 & 0 & 0 \\ \frac{-\nu}{E} & \frac{1}{E} & \frac{-\nu}{E} & 0 & 0 & 0 \\ \frac{-\nu}{E} & \frac{-\nu}{E} & \frac{1}{E} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{2(1+\nu)}{E} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{2(1+\nu)}{E} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{2(1+\nu)}{E} \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{zx} \end{Bmatrix}$$

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{zx} \end{Bmatrix} = \begin{bmatrix} \lambda + 2G & \lambda & \lambda & 0 & 0 & 0 \\ \lambda & \lambda + 2G & \lambda & 0 & 0 & 0 \\ \lambda & \lambda & \lambda + 2G & 0 & 0 & 0 \\ 0 & 0 & 0 & G & 0 & 0 \\ 0 & 0 & 0 & 0 & G & 0 \\ 0 & 0 & 0 & 0 & 0 & G \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{Bmatrix}$$

(1) Failure

(2)

(3)

(4)

1) Road

2) Foundation

Multistage

I	σ_3	σ_1
II	σ_3	σ_1
III	σ_3	σ_1

1) Draw joint failure

2) He gave us

$$\tau = S_i + \sigma \tan \phi$$

for intact rock.

For failure occurs on the joint $\beta_1 < \beta < \beta_2$

For failure occurs on the intact $\beta < \beta_1$

$\beta > \beta_2$

Disadvantage of overcoring:

1. The linear dependence of the stresses upon, the one elastic const. t.

2. Large drill cores to make the rock not crack.