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 $\sigma < \sigma_T$

$\Downarrow$

$$\text{If} \quad \left.\begin{array}{l} \beta > \dfrac{2\beta_1}{2} \\[2ex] \quad < \dfrac{2\beta_2}{2} \end{array}\right\} \text{if sliding}$$

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 $\psi_1 + \psi > \psi_2$

Note: The value of β_1 & β_2 are function of

β_1 & β_2 = f (σ_1, σ_3, ϕ_R, $S_{R,}$ S_J, ϕ_u, i, σ_T

Failure is due to sliding along discontinuity

Failure through intact rock.

Course pack   Fig. 6.6   pp. 53
                      6.7

H.W. # 7

<u>Water Pressure affects in Discontinuous Rock</u>:

$\sigma' = \sigma_T - p_w$

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 $\rightarrow \Delta u$

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 $\delta = \frac{\sigma_n}{K_n}$

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

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

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

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

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 <u>Plane Sliding</u>:

a) <u>Kinematics</u>

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°W plunging 50°
600 = acting in the direction S40°W plunging 20°

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°

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

3. Plot direction of the resultant on stereonet.

S50°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

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

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

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

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> $$F.S. = \frac{\tan \phi_{available}}{\tan \phi_{reqd.}} = \frac{\tan 25°}{\tan \phi_{reqd.}} = 2.5$$

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

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

$|B| = 15$ Tons $\quad 49.5^o$ 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: $A = \frac{H - Z}{\sin \delta} * 1$

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 = $U = \gamma_w \frac{Z_w}{2} \cdot A = \frac{1}{2} \gamma_w Z_w A.$

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.

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

For Case #1:

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

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

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 $\hat{D}$.

∴ 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 → 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

<u>Geometry:</u>

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 $\psi_{fi} > \psi_I > \phi$

<u>Forces:</u>

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

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

Fig. 96:

ξ : angle between joints

β : angle of tilt of line of intersection

Note: 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.

Ex.	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

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

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

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
∴ 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

<u>Vertical</u>

<u>Horizontal</u>

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

Inclined

<u>Fig. 9.8</u> not intact rock

resultant can't go
outside ϕ_J

$\Rightarrow$ slippage $\Rightarrow$ realignment of stresses

Close Form Solution

1. Resolve Q & P

// & ⊥ to pidding planes

1977 Bray

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

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

- 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) ∴ 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: $\tau_{res.} \simeq \dfrac{q_u}{20}$

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 $\ell = \ell_1 = \dfrac{P}{2\pi a\ \tau_{all}}$

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 $\sigma_Z\ )_{@\ Z = \ell 2} \le q_{all}$, 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
   $\sigma)_{@\ \ell} \le q_{all}$ and $\tau \le \tau_{all}$

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

σ_3 =

σ_1 = $\frac{\Delta V}{V} = \varepsilon_1 = \varepsilon_2 + \varepsilon_3$

ΔL_1

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

L_o

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

ε_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

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

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

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

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

- $V_p^2 = E/\delta$

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

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

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

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

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

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

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

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

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

(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.