GEOTECHNICAL ENGINEERING FORMULAS

A handy reference for use in geotechnical analysis and design

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1. SOIL CLASSIFICATION

1.1 USCS: Unified Soil Classification System

Coarse Grained soils have less than 50% passing the # 200 sieve:

Symbol	**Passing the #200**	**Cu=** $\frac{D_{60}}{D_{30}}$	**Cc =** $\frac{(D_{30})^2}{D_{10} \times D_{60}}$	**Soil Description**
GW	< 5%	4 or higher	1 to 3	Well graded gravel
GP	< 5%	Less than 4	1 to 3	Poorly graded gravel
GW-GM	5 to12%	4 or higher	1 to 3 but with <15% sand	Well graded gravel with silt
GW-GM	5 to12%	4 or higher	1 to 3 but with ≥15% sand	Well graded gravel with silt and sand
GW-GC	5 to12%	4 or higher	1 to 3 but with <15% sand	Well graded gravel with clay or silty clay
GW-GC	5 to12%	4 or higher	1 to 3 but with ≥15% sand	Well graded gravel with clay and sand
GC	>12%	N/A	N/A,<15%sand	Clayey Gravel
GC	> 12%	N/A	N/A,>15%sand	Clayey Gravel with sand
GM-GC	>12%	N/A	N/A,<15%sand	Clayey Silt with gravel
GM-GC	>12%	N/A	N/A,≥15%sand	Clayey Silt with sand
SW	< 5%	6 or higher	1 to 3	Well graded sand
SP	< 5%	Less than 6	1 to 3	Poorly graded sand
SM	>12%	N/A	N/A	Silty Sand or Sandy Silt
SC	>12%	N/A	N/A	Clayey Sand or Sandy Clay
SC-SM	>12%	N/A	N/A	Silty Clay with Sand

Where:

Cu = Uniformity Coefficient; gives the range of grain sizes in a given sample. Higher Cu means well graded.

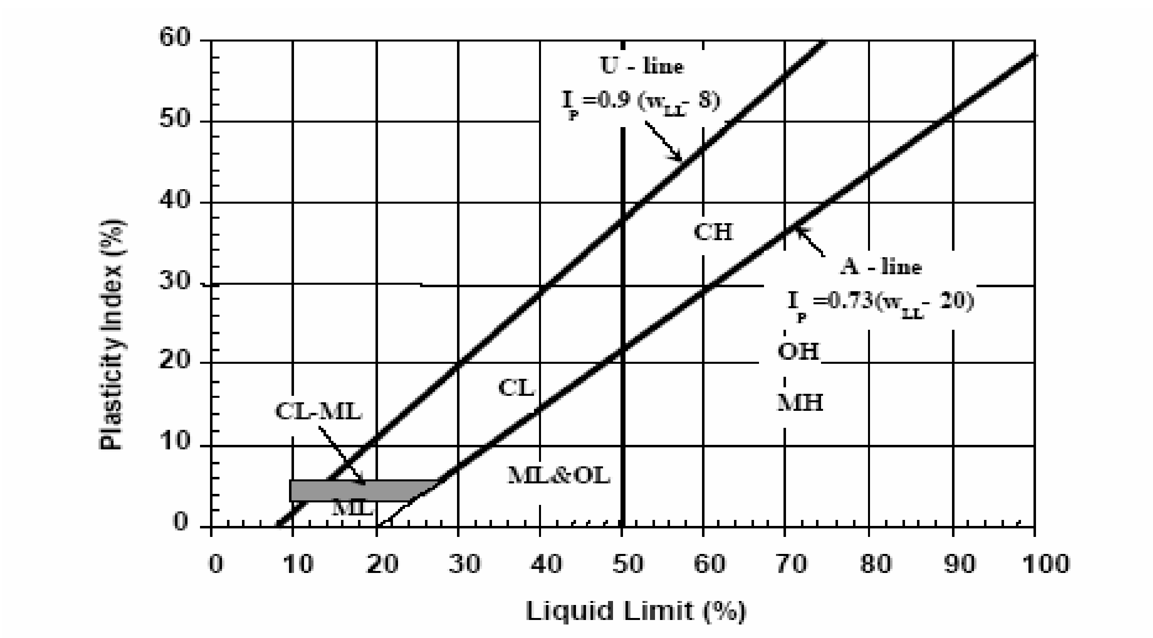
Cz = Coefficient of Curvature is a measure of the smoothness of the gradation curve. Usually less than 3.

D10, D3, & D60 are the grain size diameter corresponding to 10%, 30% and 60% passing screen.

1.1.1 Relative Density of Cohesionless Soils:

SPT or N value	Relative Density	% Relative Density
0 – 3	Very loose	0 – 15
4 – 10	Loose	15 – 35
11 – 30	Medium dense	35 – 65
31 – 50	Dense	65 -85
> 50	Very dense	85 - 100

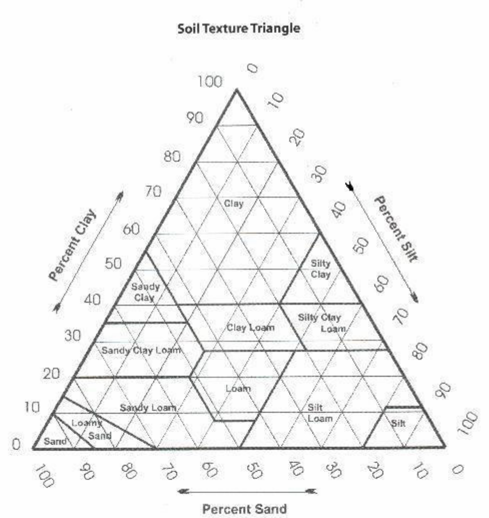
1.1.2 Fine Grained(Cohesive) Soil Charts using the USCS System:



1.1.3 Consistency of Fine Grained Soils:

SPT or N value	Cohesion, C or Su	Consistency
< 2	< 500 psf	Very soft
2 – 4	500 – 1000 psf	Soft
5 – 8	1000 – 2000 psf	Firm
9 – 15	2000 – 4000 psf	Stiff
16-30	4000 – 8000 psf	Very stiff
>30	> 8000 psf	Hard

1.2 USDA Soil Classification System

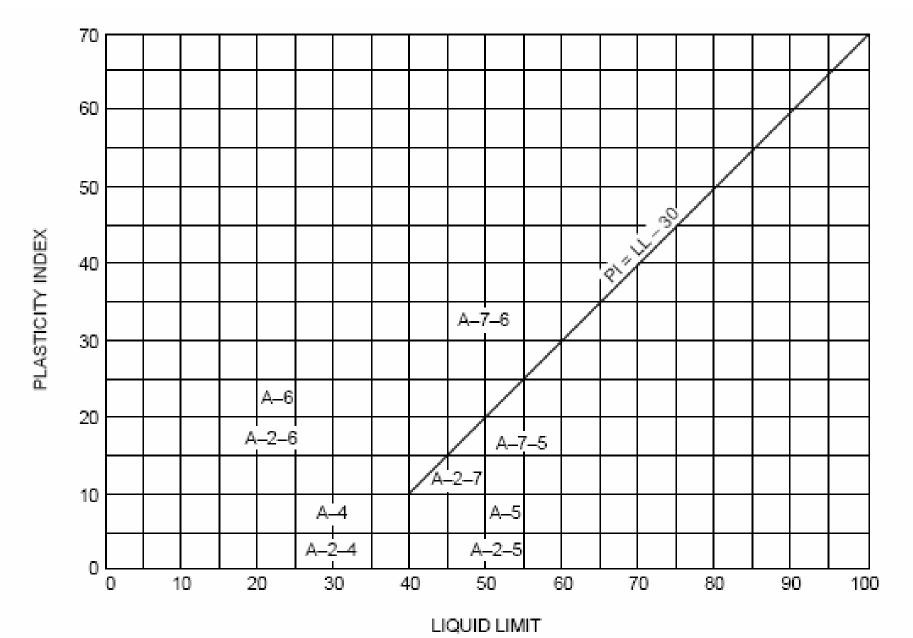


The percent SAND,SILT,and CLAY lines are drawn and their intersection gives the soil classification.

1.3 AASHTO Soil Classification System:

CLASSIFICATION OF HIGHWAY SUBGRADE MATERIALS (With suggested subgroups)											
General Classification	Granular Materials (35% or less passing No. 200)							Silt-Clay Materials (More than 35% passing #200)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, Percent Passing:											
No. 10	0-50										
No. 40	0-30	0-50	51-100								
No. 200	0-15	0-25	0-10	0-35	0-35	0-35	0-35	36-100	36-100	36-100	36-100
Characteristics of fraction passing # 40:											
Liquid Limit				0-40	41+	0-40	41+	0-40	41+	0-40	41+
Plasticity Index	0-6		N.P.	0-10	0-10	11+	11+	0-10	0-10	11+	11+
Group Index	0		0	0		0-4		0-8	0-12	0-16	0-20
Usual Types of Significant Constituent Materials	Stone Fragments, Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good					Fair to Poor					

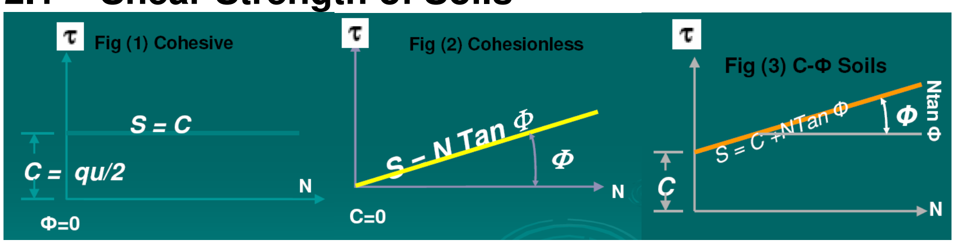
Cohesive soils classification in AASHTO System:



2. PHASE RELATIONSHIP EQUATIONS:

Dry Unit Weight, γd	Bulk or Wet or Total Unit Weight, γm or γw or γt or γ	Saturated Unit Weight, γs or γsat
$\gamma_d = \dfrac{\gamma}{1+w}$	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	$\gamma_{sat} = \dfrac{(G_s+e)\gamma_w}{1+e}$
$\gamma_d = \dfrac{G_s\gamma_w}{1+e}$	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	$\gamma_{sat} = \left(\dfrac{e}{w}\right)\left(\dfrac{1+w}{1+e}\right)\gamma_w$
$\gamma_d = \dfrac{eS\gamma_w}{(1+e)w}$	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	$\gamma_{sat} = [(1-n)G_s+n]\gamma_w$
$\gamma_d = G_s\gamma_w(1-n)$	$G_s\gamma_w(1-n)(1+w)$	$\gamma_{sat} = \gamma_d + \left(\dfrac{e}{1+e}\right)\gamma_w$

2.1 Shear Strength of Soils



S = C

S= NTanΦ

S = C + N TanΦ

2.2 Bearing Capacity of Soils

Hansen B.C. Factors:

Ø	Nc	Nq	Nγ
0	5.10	1.00	0.00
4	6.19	1.43	0.05
8	7.53	2.06	0.22
12	9.28	2.97	0.63
16	11.63	4.34	1.43
20	14.83	6.40	2.95
24	19.32	9.60	5.75
26	22.25	11.85	7.94
28	25.80	14.72	10.94
30	30.14	18.40	15.07
32	35.49	23.18	20.79
34	42.16	29.44	28.77
36	50.59	37.75	40.05
38	61.35	48.93	56.18
40	75.32	64.20	79.54

Terzaghi B.C. Factors

ϕ'	N_q	N_c	N_γ
0	1.00	5.70	0.0
2	1.22	6.30	0.2
4	1.49	6.97	0.4
6	1.81	7.73	0.6
8	2.21	8.60	0.9
10	2.69	9.60	1.2
12	3.29	10.76	1.7
14	4.02	12.11	2.3
16	4.92	13.68	3.0
18	6.04	15.52	3.9
20	7.44	17.69	4.9
22	9.19	20.27	5.8
24	11.40	23.36	7.8
26	14.21	27.09	11.7
28	17.81	31.61	15.7
30	22.46	37.16	19.7
32	28.52	44.04	27.9
34	36.50	52.64	36.0
35	41.44	57.75	42.4
36	47.16	63.53	52.0
38	61.55	77.50	80.0
40	81.27	95.66	100.4
42	108.75	119.67	180.0
44	147.74	151.95	257.0
45	173.29	172.29	297.5

Allowable Gross Bearing Capacity- 9 Equations

In Cohesionless(granular) Soils

qu = γD(Nq) + 0.6γ R(N γ)--for circular footings

qu = γD(Nq) + 0.4γ B(N γ)-- for square or rectangular footings

qu = γD(Nq) + 0.5γ B(N γ)—for continuous footings

In Cohesive (clayey) Soils

qu = 1.3 C(Nc) + γD--for circular footings

qu = CNc (1+ (0.3B/L)) + γD-- for square or rectangular footings

qu = CNc + γD-—for continuous footings

In Mixed soils (C-Φ)

qu = 1.3C(Nc) +γD(Nq) + 0.6γ R(N γ)--for circular footings

qu = CNc (1+ (0.3B/L)) + γD(Nq) + 0.4 γB(N γ) - for sq/rect. footings

qu = CNc + γDNq + 0.5 γB(N γ) -—for continuous footings

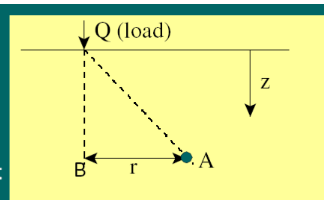
Cohesion 1st term | Surcharge 2nd term | Friction 3rd term

Note:If Df/B > 1, terzaghi's B.C. factors do not apply. Use Hansen's B.C. factors. For example, if depth of footing (Df) is 3 ft but footing width (B) is 2.75 ft.

3. STRESSES IN SOILS

3.1 Various Loading Conditions:

1) Stress due to concentrated or a Point load



Δσv = (Q/Z^2) * [3 /(2π[1+(r/z)^2]5/2)] at point A

The [second term] is IB-Boussinesq's influence coefficient and you may Plot r/z versus IB

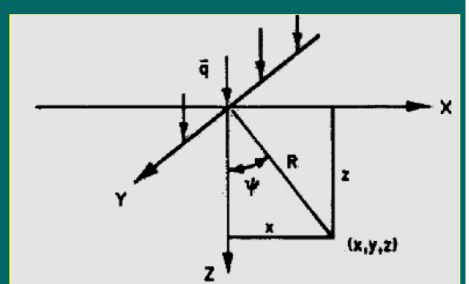
Δσv = 0.4775 (Q/Z^2) at point B

NOTE:

This assumes that the soil profile is isotropic, homogeneous, elastic half-spaced material. Since soil properties vary in direction, is comprised of more than one type and is layered, Boussinesq's formula is Conservative But useful for getting upper bound solution.

2) Stress due to a line load:

Δσv = 2q(Z^3) / π(R^4)
q = line load in k/ft or similar
R = (x^2 + z^2)^0.5
Ψ = Angle line R makes with the vertical axis

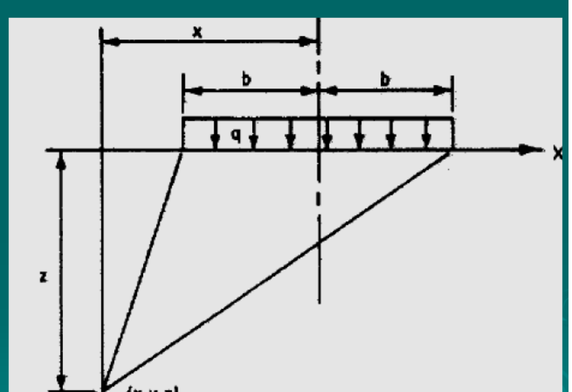


Strip

3) q = line load in ksf or similar
Δσv = q/π (α+ sinα(cos(α+2ß)

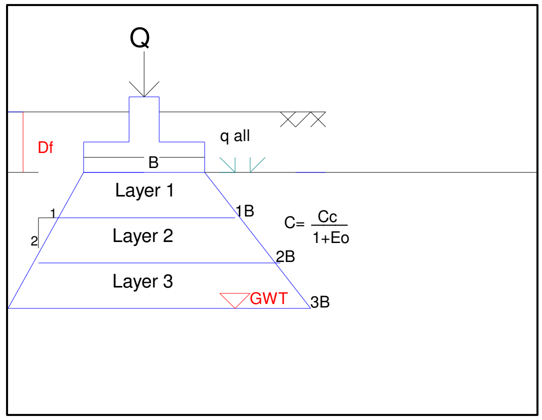
Where:
α = Tan^-1 (x+b) / z) - ß
ß = Tan^-1 (x-b) / z)

α & ß are in Radians



4. SHALLOW FOUNDATIONS

4.1 Conventional Footings



4.11Geotechnical Analysis

qall = Q / Bx1 for Continuous Footings
qall = Q / BxL for Rectangular Footings
qall = Q / BxB for Square Footings
qall < qu / 3 from Bearing Capacity Calculations
e < B/6, where e=eccentricity
Df > 1.0 ft minimum
Df > frost depth
Df > setback distance for footings on slope
Df > scour depth
Df > high moisture variations depth(expansive soils)

4.12 Structural Design:

Given: A Continuous footing with $\gamma m = 100$ pcf, Df = 5 ft, qall = 4,000 psf, D.L=22 k/ft, L.L.=12 k/ft, f'c=3 ksi, fy= 60 ksi. Design the footings using the ACI code:

1.) Effective soil pressure: Assume total depth of footing = 19 in.
Weight of footing = (19)(150)/12 = 237.5 lb./ft.^{2}
Weight of soil = (5-19/12)(100) = 341.7 lb/ft.^{2}
q_e = 4000-238-342 = 3420 lb./ft.^{2} = 3.42 k/ft.^{2}

2.) Width of footing = , **use 10 ft. footing**.

3.) Net upward pressure = P_u / Area
$P_u = 1.2D + 1.6L = (1.2)(22) + (1.6)(12) = 45.6$ k/ft.
$q_u = 45.6 / (10\cdot 1) = 4.56$ k/ft.

4.) Check one-way shear: $d = 19 - 3.5 = 15.5$ in.

$$V_u = q_u\left(\frac{L}{2}-\frac{a}{2}-d\right) = 4.56\left(\frac{10}{2}-\frac{12}{2\cdot 12}-\frac{15.5}{12}\right) = 14.63 \text{ kip}$$

$$d = \frac{V_u}{\phi 2\sqrt{f_c'}b} = \frac{14.63\cdot 1000}{0.75\cdot 2\sqrt{3000}\cdot 12} = 14.8 \text{ in.} < 15.5 \text{ in.}$$

use actual d = 15.5 in.

5.) Calculate $B.M.$ and A_s :

$$M_u = \frac{q_u}{2}\left(\frac{L}{2}-\frac{a}{2}\right)^2 = \frac{4.56}{2}\left(\frac{10}{2}-\frac{12}{2\cdot 12}\right)^2 = 46.17 \text{ k.ft.}$$

, Assume a =1.5 in.

$$A_s = \frac{M_u}{\phi f_y\left(d-\frac{a}{2}\right)} = \frac{46.17\cdot 12}{0.9\cdot 60\left(15.5-\frac{1.5}{2}\right)} = 0.7 \text{ in.}^2$$

Check

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.7\cdot 60}{0.85\cdot 3\cdot 12} = 1.364 \text{ in., o.k.}$$

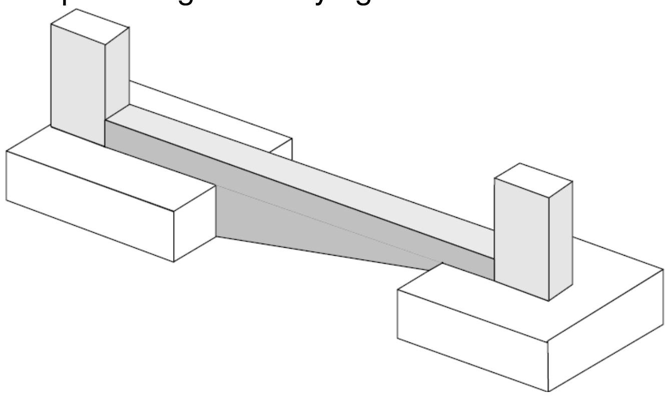
$A_{s(min)} = 0.0018bh = (0.0018)(12)(19) = 0.41$ in.^{2} < 0.70 in.^{2}
Use #7 bars @ 9 in. (A_s = 0.80 in.^{2})

6.) l_{dav} = available development length = (10· 12/2) - (12/2) - 3 = 51 in.
Required l_d = 48 in. < 51 in. (l_d from Table 7.2, chapter 7)

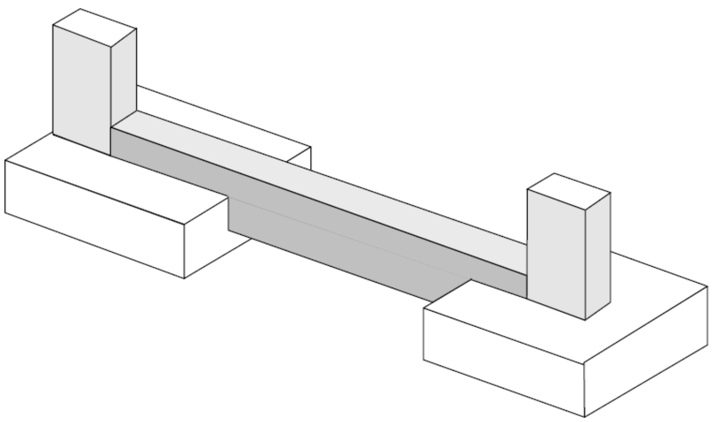
7.) Longitudinal reinforcement = $A_{s(min)}$ = 0.41 in.^{2}, **use #5 bars @ 9 in.** (A_s = 0.41 in.^{2})

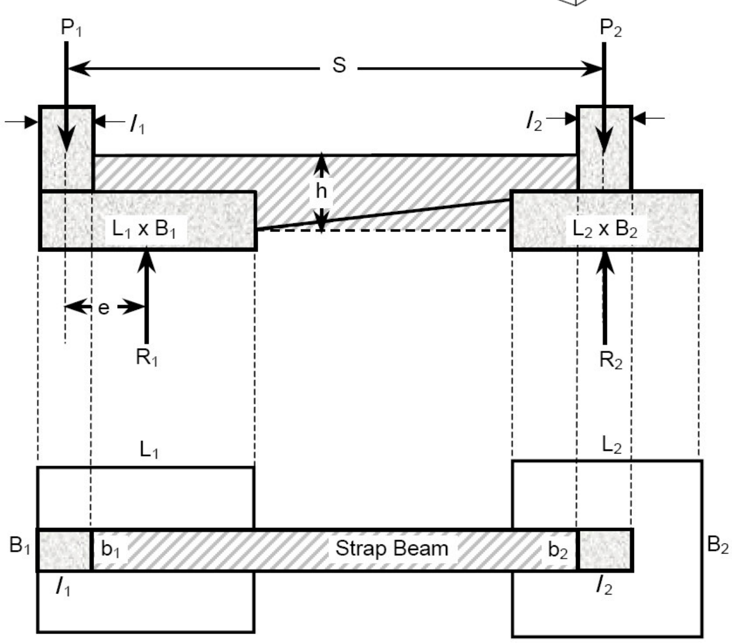
4.2 Strap or Cantilever Footings:

Strap Footing with varying beam thickness



Strap Footings with constant beam thickness





DIMENSION FOOTINGS (Determine L_1, B_1, L_2 and B^2)

Allowable load $P = P_1 + P_2$

Ultimate load $P_u = [1.4DL_1 + 1.7LL_1] + [1.4DL_2 + 1.7LL_2]$

Ultimate ratio $r_u = \dfrac{P_u}{P}$, Ultimate applied pressure $q_u = q_a \times r_u$

$\Sigma M_{col.\,2} = 0$

$R_1 (S - e) - P_{u1} S = 0$ .... (1)

$\Sigma M_{R1} = 0$

$P_{u2} (S - e) - R_2 (S - e) - P_{u1} e = 0$ .... (2)

$\Sigma F = 0$

$P_{u1} + P_{u2} - R_1 - R_2 = 0$ .... (3)

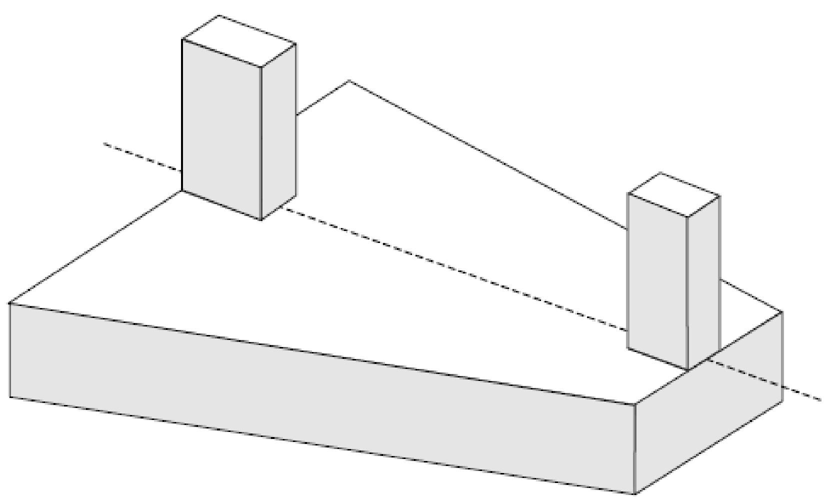
$q_u = r_u q_a$

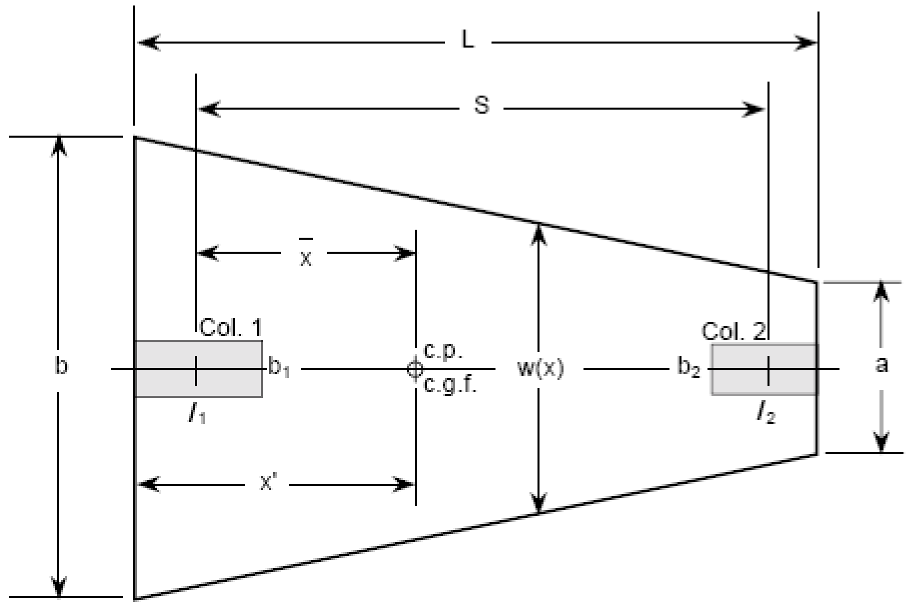
Footing 1: $L_1 = 2 \times \left(e + \dfrac{l_1}{2}\right)$ and $B_1 = \dfrac{R_1}{q_u L_1}$

Footing 2: let $k_2 = \dfrac{L_2}{B_2}$ (1.0 means footing 2 is square)

$B_2 = \sqrt{\dfrac{R_2}{k_2 q_u}}$ and $L_2 = k_2 B_2$

4.3 Trapezoidal Footings:





Allowable load $P = P_1 + P_2$

Ultimate load $P_u = [1.4DL_1 + 1.7LL_1] + [1.4DL_2 + 1.7LL_2]$

Ultimate ratio $r_u = \frac{P_u}{P}$, Ultimate applied pressure $q_u = q_a \times r_u$

$\Sigma M_{col\ 1} = 0 \qquad \bar{x} = \frac{P_{u2} \times s}{P_u}$ and $x' = \bar{x} + \frac{l_1}{2}$

For a trapezoidal solution, $\frac{L}{3} < x' < \frac{L}{2}$

For a trapezoidal solution, $\frac{L}{3} < x' < \frac{L}{2}$

From trapezoidal geometry,

$$A = \frac{a+b}{2} L \qquad \text{where } A = \text{Area} = \frac{P_u}{q_u}$$

and $$x' = \frac{L}{3}\left(\frac{2a+b}{a+b}\right)$$

From these equations, we solve for a and b

5. SOIL CONSOLIDATION EQUATIONS

5.1 Instant Settlement of footings:

In Continuous footings: $\Delta H = 8q(B^2) / Kv(B+1)^2$
In Square footings: $\Delta H = 4q(B^2) / Kv(B+1)^2$
Where:
Kv = modulus of subgrade reaction in Tons per cubic foot(Ton/ft^3)
B = footing width in feet, B is less than 20'
q = applied stress at base of footings in Tons per square foot

Kv = 50 -80 loose cohesionless soils
Kv = 80 – 150 in medium dense soils—most common value in design
Kv= 150 – 230 in Dense soils &
Kv = 230 – 300 in very dense soils

or

$$S_i = C_s q B \left(\frac{1-\nu^2}{E_u}\right)$$

S_i = immediate settlement of a point on the surface
C_s = shape and rigidity factor
q = equivalent uniform stress on the footing (total load/footing area)
B = characteristic dimension of the footing
ν = Poisson's ratio
E_u = undrained elastic modulus (Young's modulus)

5.2 Primary Consolidation:

$S=(Cc/1+eo)H \times Log(\sigma o + \Delta q)/\sigma o$

or

$$s_c = \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma_o' + \Delta q}{\sigma_o'}\right)$$

5.3 Overconsolidated Soils

Settlement of Overconsolidated soils Case 1:($\sigma f < Pc$)

$\Delta H = (Cr/1+eo)H \times Log\ Pc/\sigma o$

or

$$s_c = \frac{C_r H}{1 + e_o} \log\left(\frac{\sigma_o' + \Delta q}{\sigma_o'}\right)$$

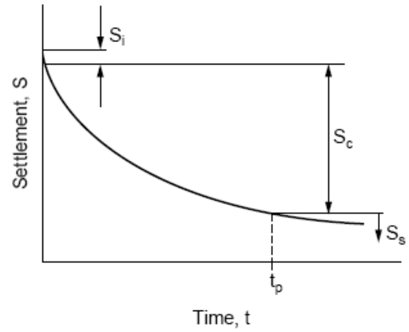
Settlement of Overconsolidated soils Case 2:($\sigma f > Pc$)

$\Delta H = (Cr/1+eo)H \times Log(Pc/\sigma o) + (Cc/1+eo)H \times Log(\sigma f/\sigma o)$

or

$$s_c = \left(\frac{H}{1 + e_o}\right)\left(C_r \log\left(\frac{P_c}{\sigma_o'}\right) + C_c \log\left(\frac{\sigma_o' + \Delta q}{P_c}\right)\right)$$

5.4 Time rate of settlement (i=immediate, c=consolidation, & s=secondary)



5.41 Coefficient of consolidation, Cv:

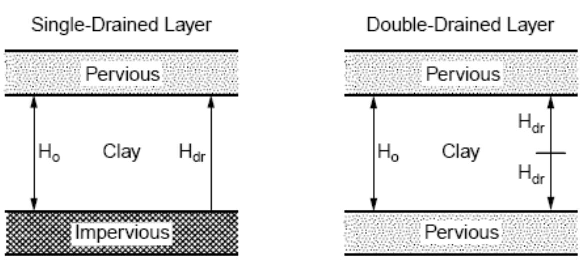
$$c_v = \frac{k(1+e_o)}{\gamma_w a_v}$$

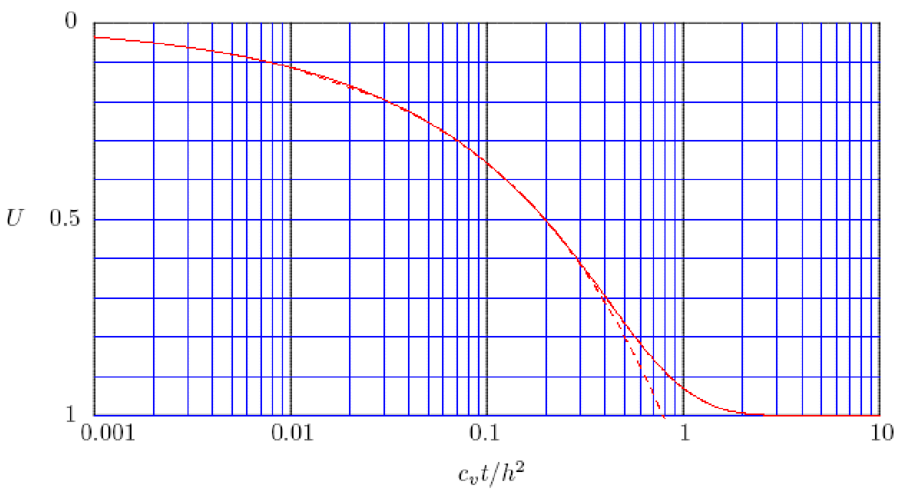
k = hydraulic conductivity
γ_w = unit weight of water
e_o = initial void ratio
$a_v = -de/d\sigma'_v$ = coefficient of compressibility

$$Z = \frac{z}{H_{dr}}$$

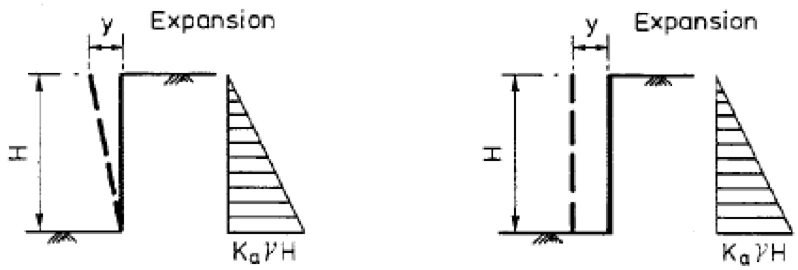
$$T = \frac{c_v t}{H_{dr}^2}$$

z = depth below top of the compressible stratum
H_{dr} = length of the longest pore water drainage path

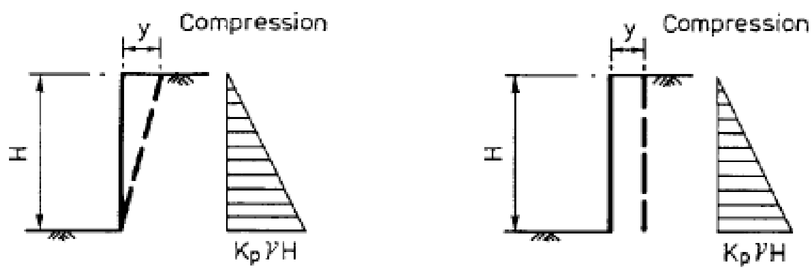




6. RETAINING STRUCTURES:

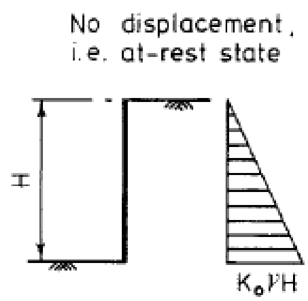


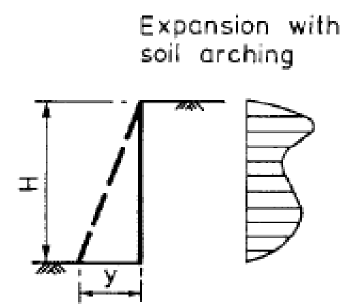
(i) Active State



(ii) Passive State

(a) Rigid Retaining Wall Free to Translate or Rotate about Its Base

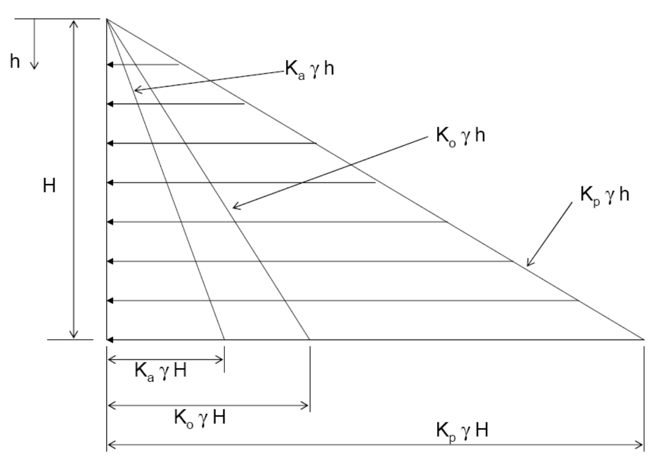




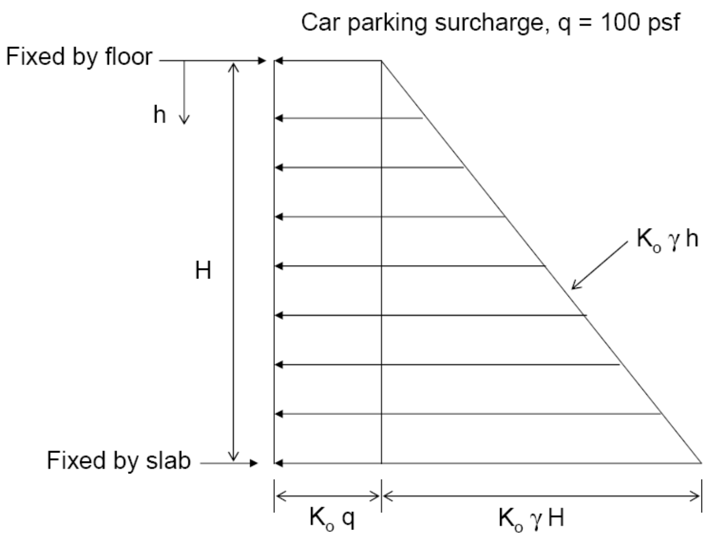
(b) Restrained Rigid Wall

(c) Top of Wall Restrained

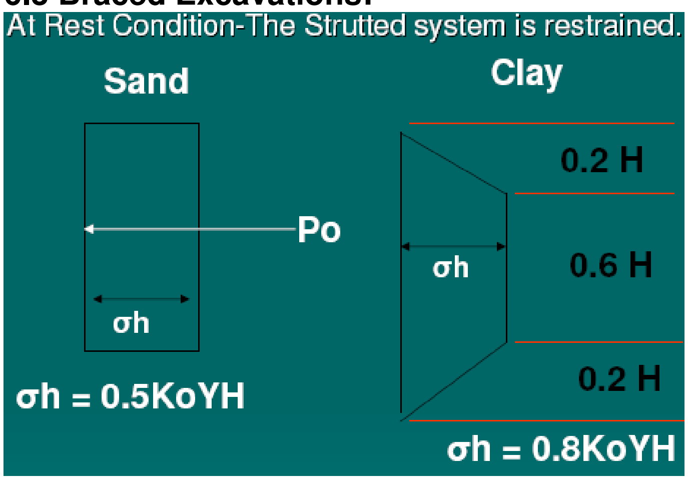
6.1 Horizontal Stresses: Active, At Rest and Passive

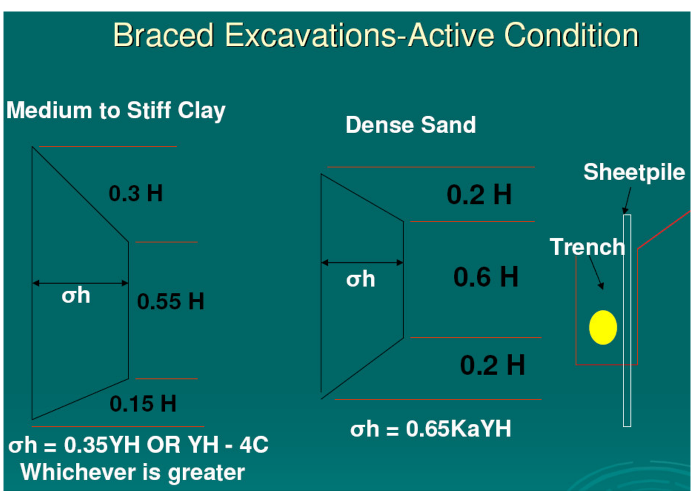


6.2 Basement Wall with surcharge:

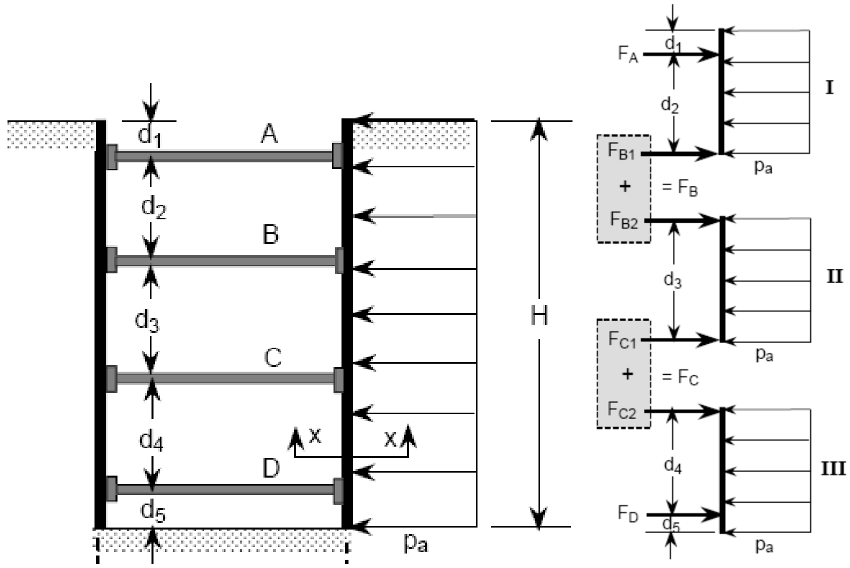


6.3 Braced Excavations:





6.4 Forces on Struts:



Note that the first strut A must be placed at a depth $d_1 < z_c$ (depth of tension crack) where $z_c = \frac{2\,c}{\gamma}$.

Forces on Struts and Selection of Section
(Designed as column, pined at both ends)

1) Draw the pressure diagram p_a
2) Assume that the sheet pile is hinged at all levels of struts
3) Calculate F_A , F_{B1}, F_{B2}, F_{C1}, F_{C2}, and F_D which are the reaction in the load distributions I, II and III.
4) The loads in the struts are calculated as:

$P_A = (F_A) \times s$

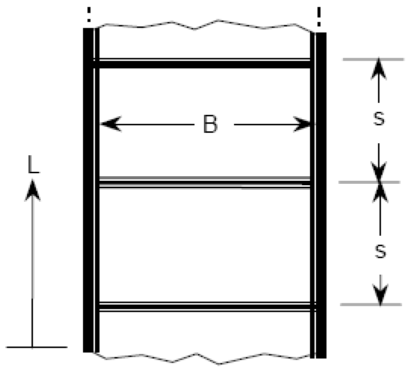
$P_B = (F_{B1} + F_{B2}) \times s$

$P_C = (F_{C1} + F_{C2}) \times s$

$P_D = (F_D) \times s$

Maximu Moment on Sheet Pile and selection of Section

1) For each of the load distributions I, II and III find M_{max} i.e. where the shear is equal to zero.
2) The design moment for the sheet pile is the maximum of step (1)
3) Calculate the section modulus $S = \frac{M_{max}}{\sigma_{all}}$ where σ_{all} = allowable stress for sheet pile
4) Select the sheet pile section based on S in Step 3



Section x-x (Plan)

<u>Maximu Moment on Wales and selection of Section</u>
(Designed as beams pined at the struts)

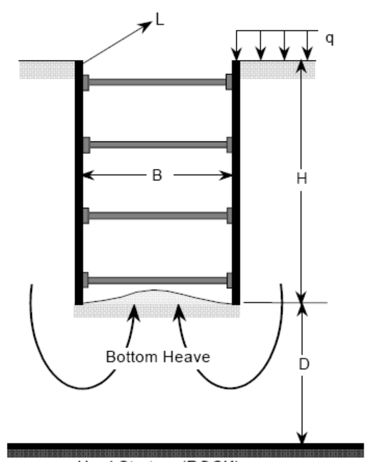
At level A: $M_{A,\,max} = \dfrac{F_A\ \left(s^2\right)}{8}$

At level B: $M_{B,\,max} = \dfrac{\left(F_{B1} + F_{B2}\right)\ \left(s^2\right)}{8}$

At level C: $M_{C,\,max} = \dfrac{\left(F_{C1} + F_{C2}\right)\ \left(s^2\right)}{8}$

At level D: $M_{D,\,max} = \dfrac{F_D\ \left(s^2\right)}{8}$

Bottom Heave Calculations:



Safety Factor against bottom heave $SF_H \geq 1.5$

If $D > 0.7\ B$

$$SF_H = \frac{1}{H}\left(\frac{5.7\ c}{\gamma - \dfrac{c}{0.7B}}\right)$$

If $D \leq 0.7\ B$

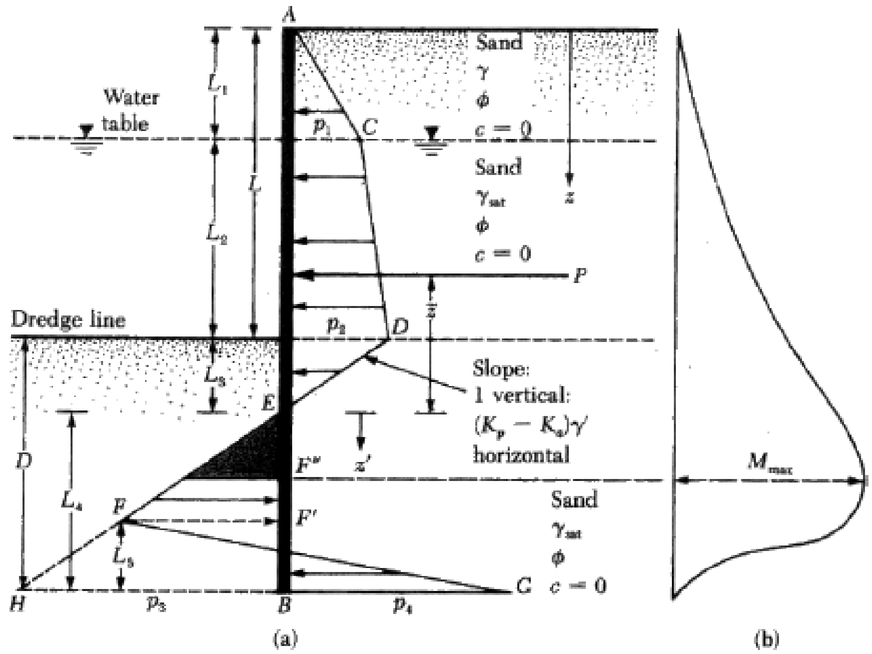
$$SF_H = \frac{1}{H}\left(\frac{5.7\ c}{\gamma - \dfrac{c}{0.7B}}\right)$$

OR

$$SF_H = \frac{c\ N_c}{\gamma\ H + q}$$

Whichever is larger

6.5 Cantilever Sheetpiles in Sand



(a) Sheet Pile in Sand with dimensions and pressure distribution
(b) Moment distribution over the sheet pile.

1. Calculate: $k_a = \tan^2\left(45 - \frac{\phi}{2}\right)$ and

   $k_p = \frac{1}{k_a}$ Note: Some designers use $k_{p(Design)} = \frac{k_p}{SF}$ where SF = 1.5 –2.0

2. Calculate: $p_1 = \gamma L_1 k_a$ and

   $p_2 = \left(\gamma L_1 + \gamma' L_2\right) \times k_a$, $\gamma' = \gamma_{SAT} - \gamma_w$

3. Calculate: $L_3 = \frac{p_2}{\left[\gamma'\left(k_p - k_a\right)\right]}$

4. Calculate: $P = \text{Area ACDE} = \frac{1}{2}p_1L_1 + p_1L_2 + \frac{1}{2}(p_2 - p_1)L_2 + \frac{1}{2}p_2L_3$

5. Calculate: $\bar{z}$ by taking moment about E of area ACDE = $P\bar{z}$

6. Calculate: $p_5 = \left(\gamma L_1 + \gamma' L_2\right) \times k_p + \gamma' L_3 \left(k_p - k_a\right)$

7. Calculate: L_4 by trial and error from the equation:

   $L_4^4 + A_1L_4^3 - A_2L_4^2 - A_3L_4 - A_4 = 0$,

   where,

   $A_1 = \frac{p_5}{\left[\gamma'\left(k_p - k_a\right)\right]}$ $A_2 = \frac{8P}{\left[\gamma'\left(k_p - k_a\right)\right]}$

   $A_3 = 6P\left[\frac{\left[2\bar{z}\gamma'\left(k_p - k_a\right) + p_5\right]}{\left[\gamma'\left(k_p - k_a\right)\right]^2}\right]$ $A_4 = P\left[\frac{\left(6\bar{z}p_5 + 4P\right)}{\left[\gamma'\left(k_p - k_a\right)\right]^2}\right]$

8. Calculate: $p_4 = p_5 + \gamma' L_4 (k_p - k_a)$, $p_3 = \gamma' L_4 (k_p - k_a)$ and

$$L_5 = \frac{(p_3 L_4 - 2P)}{(p_3 + p_4)}$$

Draw the sheet pile (similar to page 1) with the estimated values in steps 1-8

9. Calculate: D = 1.3 to $1.6(L_3 - L_4)$

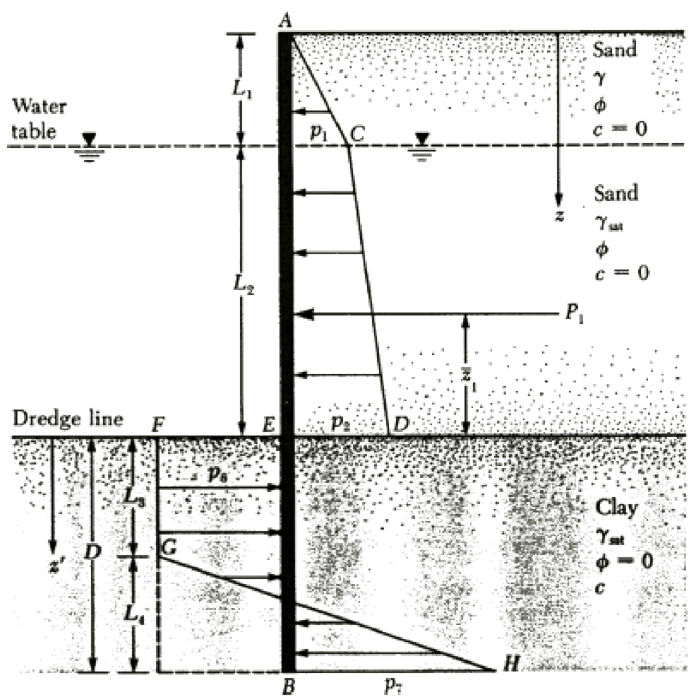
10. Calculate: $z' = \sqrt{\frac{2P}{[\gamma'(k_p - k_a)]}}$

11. Calculate: $M_{max} = P(\bar{z} + z') - \left[\frac{1}{2}\gamma' z'^2 (k_p - k_a)\right]\left(\frac{1}{3}\right)z'$

12. Calculate: $S = \frac{M_{max}}{\sigma_{all}}$

where S = Minimum section modulus of sheet pile
σ_{all} = allowable stress for sheet pile

6.6 Cantilever Sheetpiles in Clay



Sheet Pile in Clay with dimensions and pressure distribution

Design Steps (refer to figure above for terms)

1. Calculate: $k_a = \tan^2\left(45 - \frac{\phi}{2}\right)$

2. Calculate: $p_1 = \gamma L_1 k_a$ and

   $p_2 = \left(\gamma L_1 + \gamma' L_2\right) \times k_a$ where $\gamma' = \gamma_{SAT} - \gamma_w$

3. Calculate: $\frac{1}{2} p_1 L_1 + p_1 L_2 + \frac{1}{2}(p_2 - p_1)\, L_2$

4. Calculate: $\overline{z}_1$ by taking moment about E of area ACDE = $P_1 \overline{z}_1$

5. Calculate: D by the following equation :

$$D^2\left[4c - \left(\gamma L_1 + \gamma' L_2\right)\right] - 2DP_1 - \frac{P_1\left(P_1 + 12c\overline{Z_1}\right)}{\left(\gamma L_1 + \gamma' L_2\right) + 2c}$$

6. Calculate: D_{actual} = 1.3 to 1.6 D

7. Calculate: $p_6 = 4c - \left(\gamma L_1 + \gamma' L_2\right)$ and $p_7 = 4c + \left(\gamma L_1 + \gamma' L_2\right)$

8. Calculate: $L_4 = \frac{\left(Dp_6 - P_1\right)}{4c}$ and

   Draw the sheet pile (similar to page 1) with the estimated values in steps 1-8

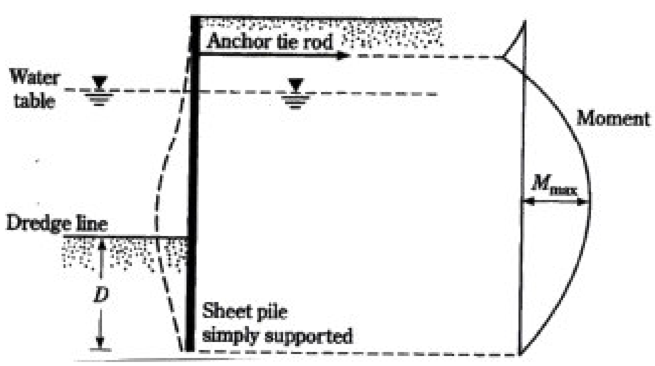
9. Calculate: $z' = \frac{P_1}{p_6}$

10. Calculate: $M_{max} = P_1\left(\overline{z_1} + z'\right) - \frac{p_6 z'^2}{2}$

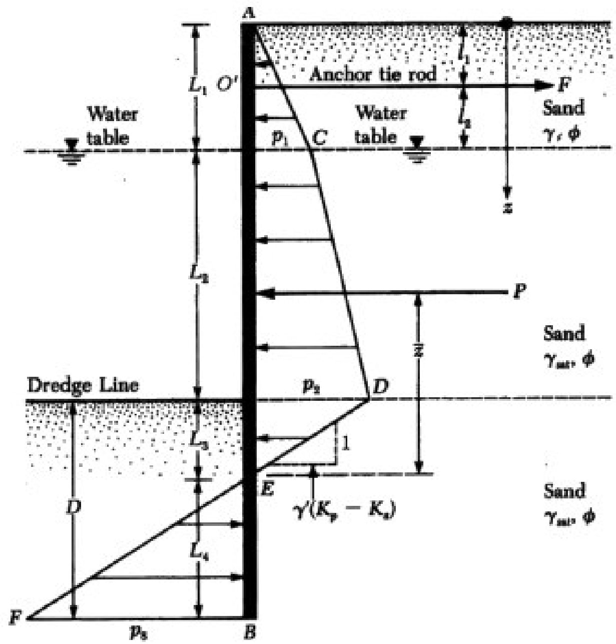
11. Calculate: $S = \frac{M_{max}}{\sigma_{all}}$

    where S = Minimum section modulus of sheet pile
    σ_{all} = allowable stress for sheet pile

6.6 Anchored Sheetpiles in Sand (Also called Bulkheads)



Deformation and moment distribution over the sheet pile.



Sheet Pile in Sand with dimensions and pressure distribution

Design Steps (refer to figure above for terms)

1. Calculate: $k_a = \tan^2\left(45 - \frac{\phi}{2}\right)$ and

   $k_p = \frac{1}{k_a}$ Note: Some designers use $k_{p(Design)} = \frac{k_p}{SF}$ where SF = 1.5 –2.0

2. Calculate: $p_1 = \gamma L_1 k_a$ and $p_2 = (\gamma L_1 + \gamma' L_2) \times k_a$, where $\gamma' = \gamma_{SAT} - \gamma_w$

3. Calculate: $L_3 = \frac{p_2}{[\gamma'(k_p - k_a)]}$

4. Calculate: P = Area ACDE = $\frac{1}{2}p_1 L_1 + p_1 L_2 + \frac{1}{2}(p_2 - p_1)L_2 + \frac{1}{2}p_2 L_3$

5. Calculate: $\bar{z}$ by taking moment about E of area ACDE = $P\bar{z}$

6. Calculate: L_4 by trial and error from the equation:

   $$L_4^3 + 1.5L_4^2(l_2 + L_2 + L_3) - \frac{3P[(L_1 + L_2 + L_3) - (\bar{z} + l_1)]}{[\gamma'(k_p - k_a)]} = 0$$

7. Calculate: $D = L_3 + L_4$ (D_{actual} = 1.3 to 1.6 D)

8. Calculate: Force in anchor rod $F = P - \frac{1}{2}[\gamma'(k_p - k_a)] \times L_4^2$

9. Calculate: $p_8 = \gamma'(k_p - k_a) \times L_4$ and

Draw the sheet pile (similar to page 1) with the estimated values in steps 1-8

10. Calculate: z_m for zero shear, hence, M_{max} by :

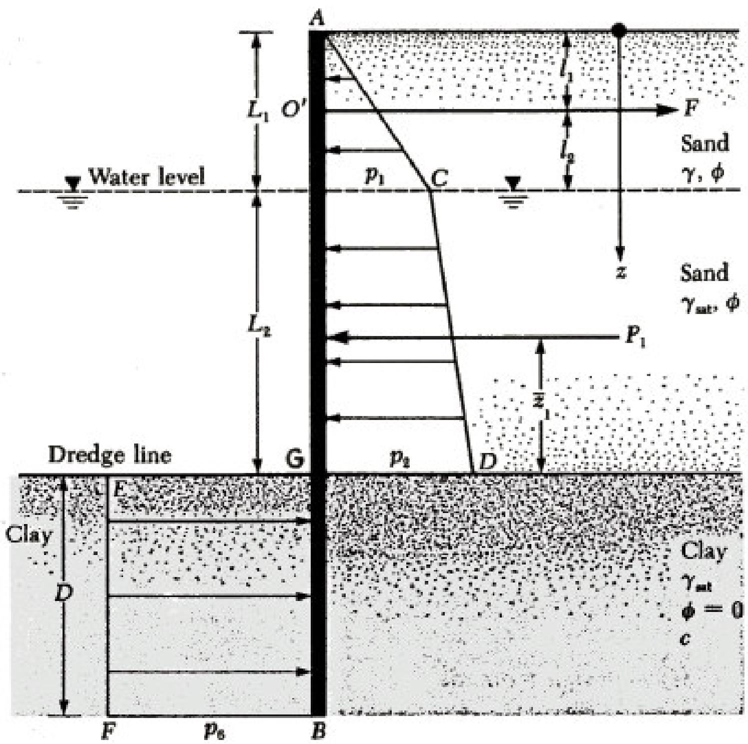
$$\frac{1}{2}p_1L_1 - F + p_1(z_m - L_1) + \frac{1}{2}k_a\gamma'(z_m - L_1)^2 = 0$$

$$L_1 < z_m < (L_1 + L_2)$$

11. Calculate: M_{max} by summing moments at a point z_m from surface.

12. Calculate: $S = \dfrac{M_{max}}{\sigma_{all}}$

6.7 Anchored Sheetpiles in Clay (Also called Bulkheads)



Anchored Sheet Pile in Clay with dimensions and pressure distribution

Design Steps (refer to figure above for terms)

1. Calculate: $k_a = \tan^2\left(45 - \frac{\phi}{2}\right)$

2. Calculate: $p_1 = \gamma L_1 k_a$ and

$$p_2 = (\gamma L_1 + \gamma' L_2) \times k_a \text{ , where } \gamma' = \gamma_{SAT} - \gamma_w$$

3. Calculate: P_1 = Area ACDG = $\frac{1}{2}p_1L_1 + p_1L_2 + \frac{1}{2}(p_2 - p_1)L_2$

4. Calculate: $\bar{z}_1$ by taking moment about G of area ACDG = $P_1\bar{z}_1$

5. Calculate: $p_6 = 4c - (\gamma L_1 + \gamma' L_2)$

6. Calculate: D from the following equation:

$$p_6D^2 + 2p_6D(L_1 + L_2 - l_1) - 2P_1(L_1 + L_2 - l_1 - \bar{z}_1) = 0$$

7. Calculate: Force in anchor rod $F = P_1 - p_6 D$

Draw the sheet pile (similar to page 1) with the estimated values in steps 1-6

8. Calculate: D_{actual} = 1.3 to 1.6 D ,

9. Calculate: z_m for zero shear, hence, M_{max} by :

$$\frac{1}{2}p_1L_1 - F + p_1(z_m - L_1) + \frac{1}{2}k_a\gamma'(z_m - L_1)^2 = 0$$

$$L_1 < z_m < (L_1 + L_2)$$

10. Calculate: M_{max} by summing moments at a point z_m from surface.

11. Calculate: $S = \frac{M_{max}}{\sigma_{all}}$

where S = Minimum section modulus of sheet pile
σ_{all} = allowable stress for sheet pile

7. PILE FOUNDATIONS

7.1 Single Piles Equations:

Q_U = Ultimate Pile Capacity

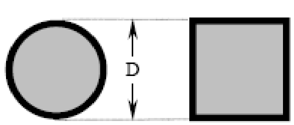
Q_p = Load-capacity of pile point

Q_s = Skin friction resistance

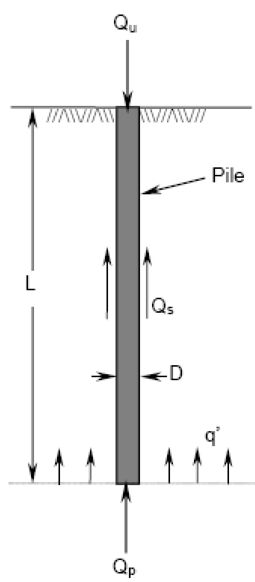
L = Pile length

q' = overburden pressure at pile tip

D = Pile dimension



Pile cross-section



A_p = Area of the pile cross-section

Load-Capacity of Pile Point

$Q_p = A_p \, q' \, N_q^*$

or

$Q_p = A_p \, q_l$

whichever is smaller

where q_l (kPa) = $50 \, N_q^* \, \tan\phi$

and N_q^* = Bearing capacity factor obtained from Fig. 1

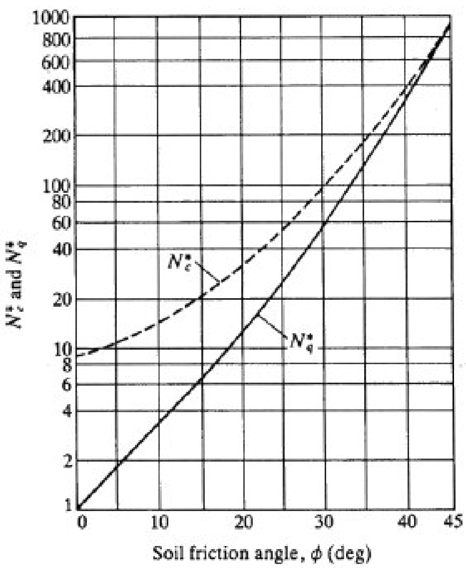


Fig. 1 Variation of N_c^* and N_q^* with φ

Skin Friction Resistance

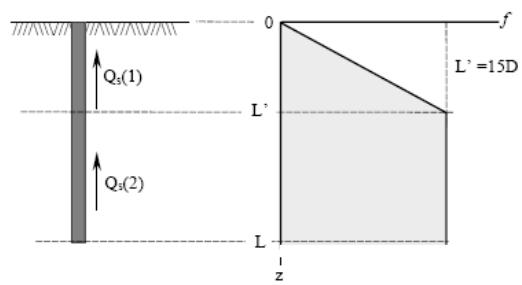


Fig. 2. Variations of skin friction with depth

$$\left.\begin{array}{l} Q_s(1) = p \, L' \, f_{av} \\ Q_s(2) = p \, (L - L') \, f \end{array}\right\} \quad Q_s = Q_s(1) + Q_s(2)$$

where

$f = K \, \sigma_v' \tan\delta$ (skin friction)

σ_V' = overburden pressure

K = lateral earth pressure = $0.5 + 0.008 \, D_r$ (D_r = relative density in percent)

δ = friction angle between soil and pile (usually 0.6φ)

p = perimeter of the pile

f_{av} = average skin friction from 0 to L = $f/2$

7.2 Group capacity of piles:

<u>Efficiency of Pile Group</u>

$$\text{Efficiency, } \eta = \frac{2\left(n_1 + n_2 - 2\right) d \; + \; 4D}{p \; n_1 \; n_2}$$

where the parameters are as defined in Fig. 3

Ultimate load capacity of pile group:

$Q_u(g) = \Sigma Q_u * \eta$

if $\eta \geq 1.0$ use $\eta = 1.0$

Allowable load capacity of pile group:

$Q_a(g) = Q_u(g)/SF$

where $2.5 \leq SF \leq 4.5$

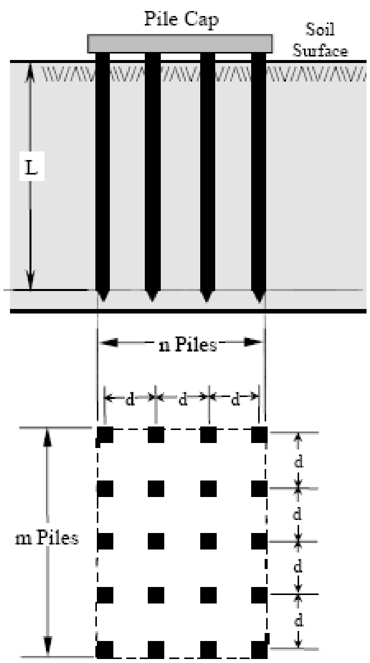


Fig. 3. Pile group

Example:

Estimate the total load that the pile group shown below can carry. Note that the every single pile is identical to that in Examples 1 and 2.

SOLUTION:

Total load that the a single pile can carry
$Q = 844.73$ kN (Examples 1 and 2)

$m = 5$, $n = 4$ and $mn = 20$ piles

$Q_T = 844.73 \times 20 = 16894.6$ kN

The efficiency of the group:

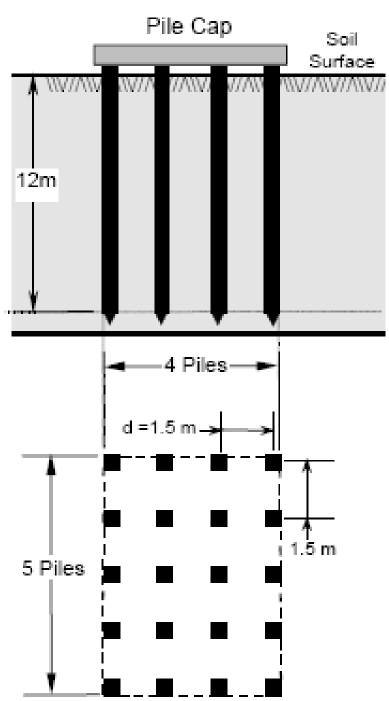
$$\eta = \frac{2\left(m \; + n \; -2\right) d \; + \; 4D}{p \; m \; n}$$

$$\eta = \frac{2\left(5 \; +4 \; -2\right)(1.5) \; + \; 4x(0.305)}{(4x0.305) \; x5 \; x4}$$

$= 0.91$ (or 91%)

The total load that can be carried by
The pile group is, therefore

$Q_{TG} = 16894.6 \times 0.91 = 15374.09$ kN



7.3 Settlement of Group Piles:

The settlement for group piles in coarse-grained soils from SPT and cone penetration tests can be estimated from:

SPT: $$\rho_{es} = \frac{360 \Delta\sigma_z I \sqrt{B_g}}{N_{cor}} \text{ (mm)} \qquad (8.44)$$

Cone $$\rho_{es} = \frac{\Delta\sigma_z I B_g}{2q_c} \qquad (8.45)$$

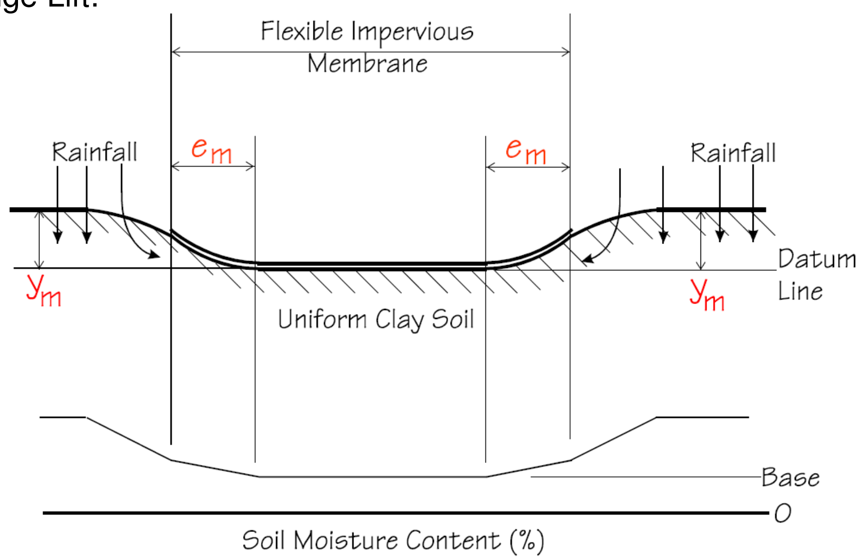
where: $I = 1 - 0.08\frac{L}{B_g} \geq 0.5$, $\Delta\sigma_z$ is the stress increase at a depth of $\frac{2L}{3}$, i.e.,

$$\left(\Delta\sigma_z \approx \frac{Q_a}{\left(B_g + \frac{2}{3}L\right)\left(L_g + \frac{2}{3}L\right)} \right)$$

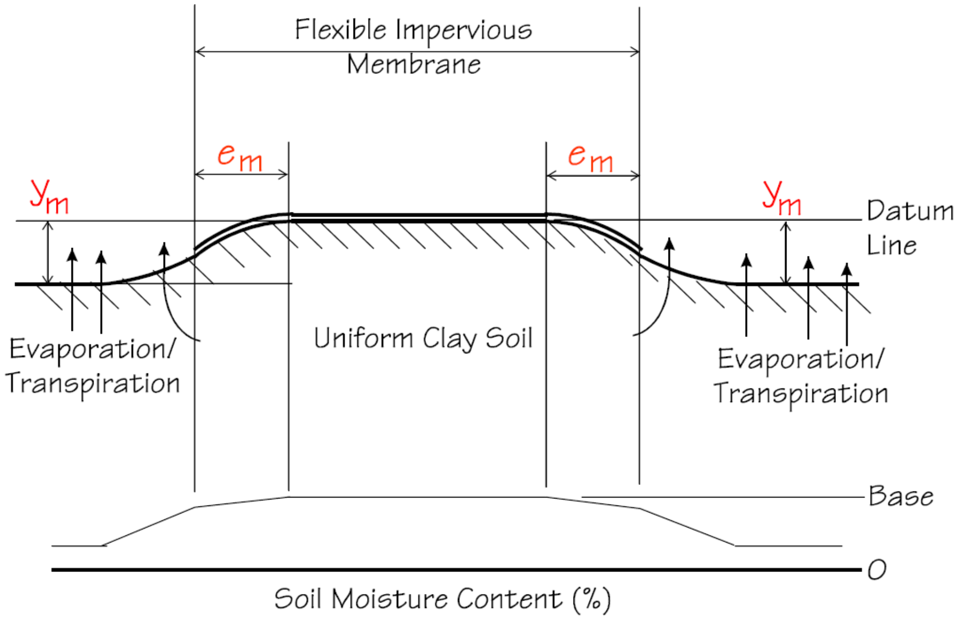
B_g and L_g are the width and length of the pile group, L is the embedded length of the pile, and q_c is the arithmetic average of the cone resistance over two pile diameters below the cone tip.

8. Post Tensioned Slabs:

Edge Lift:



Center Lift:



Edge Lift

- Soils are wetter at slab edge than at any point inside slab edge.

Center Lift

- Soils are drier at slab edge than at any point inside slab edge.

Edge Moisture Variation Distance e_m

- Thornthwaite Moisture Index (climate)
- Soil Permeability
- Vegetation

Unrestrained Differential Swell y_m

- Properties (activity) of clay
- Depth of clay (active zone)
- Soil suction

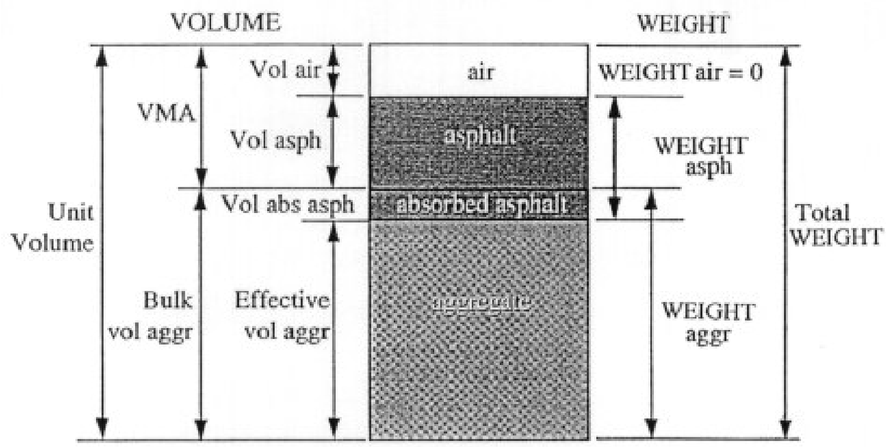
One set of e_m & y_m values established for each swell mode (edge and center lift)

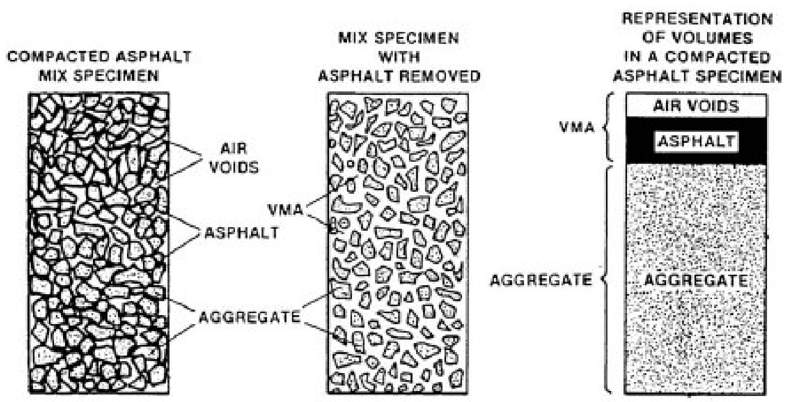
Design cannot be done without these parameters

	Edge Lift	Center Lift
e_m	2.0 ft	5.0 ft
y_m	0.75 in	3.0 in

The Structural Engineer also needs Kv (given in immediate settlement section), effective PI(pp 138 of Geotechnical DVD book) and other climatic constants that are from building codes(given).

9. Asphalt Mix Design:





LOW STABILITY

Causes	Effects
Excess binder in HMA	Washboarding, rutting, and flushing or bleeding
Excess medium size sand in HMA	Tenderness during rolling and for a period after construction, and difficulty in compacting
Rounded aggregate, little or no crushed surfaces	Rutting and channeling

POOR DURABILITY

Causes	Effects
Low binder content	Dryness or ravelling
High void content through design or lack of compaction	Early hardening of binder followed by cracking or disintegration
Water susceptible (hydrophillic) aggregate in HMA	Films of binder strip from aggregate leaving an abraded, ravelled, or mushy pavement

MIX TOO PERMEABLE

Causes	Effects
Low binder content	Thin binder films will cause early aging and ravelling
High void content in design HMA	Water and air can easily enter pavement causing oxidation and disintegration
Inadequate compaction	Will result in high voids in pavement leading to water infiltration and low strength

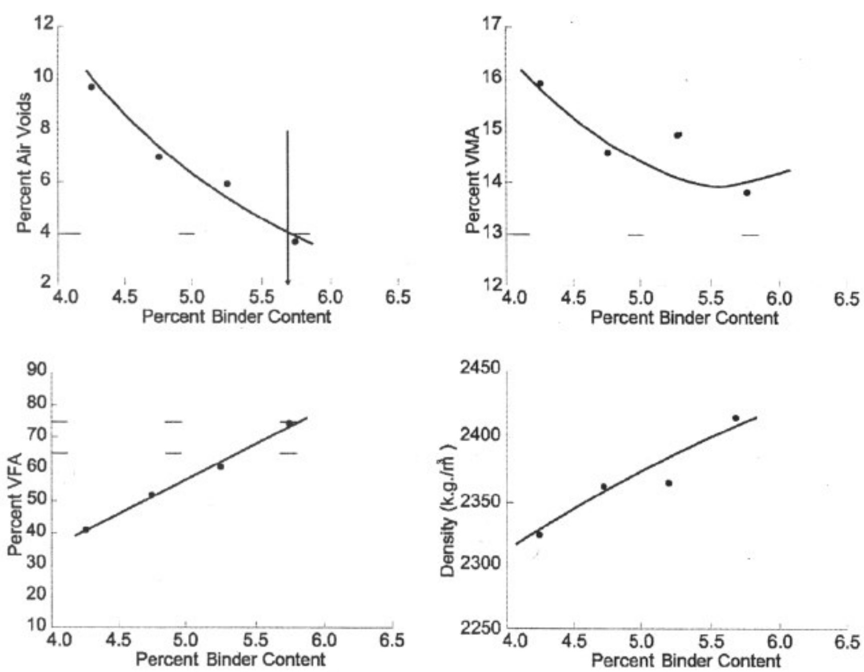
POOR FATIGUE RESISTANCE

Causes	Effects
Low asphalt binder content	Fatigue cracking
High design voids	Early aging of binder followed by fatigue cracking
Lack of compaction	Early aging of binder followed by fatigue cracking
Inadequate pavement thickness	Excessive bending followed by fatigue cracking

AC Mix design Formulas:

$$\%G_{mm} = 100x\frac{G_{mb}xh_d}{G_{mm}xh_i}$$

G_{mb} = bulk specific gravity at N_{des}
G_{mm} = theoretical maximum specific gravity at N_{des}
h_d = height of specimen at N_{des}
h_i = height of specimen at N_{ini}



When weighing in Water:

$$\text{Maximum Specific Gravity } (Gmm) = \frac{A}{A-(C-B)}$$

where:
A = weight of oven dry sample in air, g
B = weight of container in water, g
C = weight of container and sample in water, g

When weighing in Air:

$$\text{Maximum Specific Gravity } (Gmm) = \frac{A}{A+D-E}$$

where:
A = weight of oven dry sample in air, g
D = weight of container filled with water at 77°F, g
E = weight of container filled with sample and water at 77°F, g

$$\text{Bulk Specific Gravity } (Gmb) = \frac{A}{B-C}$$

where:
A = weight of specimen in air, g
B = weight of surface-dry specimen in air, g
C = weight of specimen in water, g

$$\text{Percent Water Absorbed by Volume} = \left(\frac{B-A}{B-C}\right) \text{ x } 100$$

Open Graded Mixtures:

$$Gmb = \frac{A}{B-E-\left(\frac{B-A}{F_T}\right)}$$

where:
A = weight of dry specimen in air, g
B = weight of dry, sealed specimen, g
E = weight of sealed specimen in water, g
(weight of absorbed water is subtracted)
Ft = apparent specific gravity of plastic sealing material at 77°F

$$\text{Water Absorption, percent} = \left(\frac{A_1-A}{A}\right)\times 100$$

$$\text{Dust Proportion} = \frac{P_{200}}{Pbe}$$

where:
P_{200} = aggregate content passing the No. 200 sieve, percent by weight of aggregate
Pbe = effective binder content, percent by total weight of mixture

$$\text{Absorbed Asphalt } (Pba) = 100 \text{ x } \left(\frac{Gse-Gsb}{Gsb\times Gse}\right) \text{ x } Gb$$

where:
Gse = effective specific gravity of aggregate
Gsb = bulk specific gravity of aggregate
Gb = specific gravity of binder

$$\text{Air Voids } (Va) = 100 \text{ x } \left(\frac{Gmm - Gmb}{Gmm}\right)$$

where:
Gmm = Maximum Specific Gravity of HMA
Gmb = Bulk Specific Gravity of HMA

$$\text{Voids in the Mineral Aggregate } (VMA) = 100 - \left(\frac{Gmb \times Ps}{Gsb}\right)$$

where:
Gmb = Bulk Specific Gravity of HMA
Gsb = Bulk Specific Gravity of aggregate
(Obtained from design mix formula)
Ps = Aggregate, percent by total weight of HMA

The percent of aggregate by total weight of HMA (Ps) is determined by subtracting the actual binder content by total weight of HMA (Pb) supplied on the design mix formula from 100.

$$Ps = 100 - Pb$$

$$\text{Voids Filled with Asphalt } (VFA) = \left(\frac{VMA - Va}{VMA}\right) \text{x } 100$$

10. Concrete Mix Design:

$$\text{Free Moisture } (\%) = \frac{Ww - Ws}{Wd} x100$$

Where: Ws = Saturated surface-dry weight.

$$\text{Absorbed Moisture } (\%) = \frac{Ww - Wd}{Wd} x100$$

Fineness modulus:

The standard size sieves are 6 inch, 3 inch, 1 1/2 inch, 3/4 inch, 3/8 inch, No. 4, No. 8, No. 16, No. 30, No. 50, and No. 100. In this series, the size of each opening, beginning with the 100-mesh sieve, is one-half that of the next larger size used. The percent of material passing the 100-mesh sieve is not used in calculating the fineness modulus. For example, the fineness modulus of a fine aggregate, such as would be used in concrete, may be as follows:

FINE AGGREGATE - SAND, GRADING A

Sieve Size	Percent Retained	Cumulative Percent Retained
3/8" (9.5 mm)	0.0	0.0
No .4 (4.75 mm)	0.0	0.0
No. 8 (2.36 mm)	12.0	12.0
No. 16 (1.18 mm)	15.0	27.0
No. 30 (0.600 mm)	32.0	59.0
No. 50 (0.300 mm)	18.0	77.0
No. 100 (0.150 mm)	13.0	90.0
No. 200 (0.075 mm)	10.0	100.0 (Not Include.)
		Total = 265.0

$$\frac{2.65}{100} = 2.65 = \text{F.M.(Ans.)}$$

Yield:

The yield of concrete produced per batch shall be calculated as follows:

$$Y = \frac{(N \times 94) + Wf + Wc + Ww}{W}$$

Where: Y = Yield of concrete produced per batch, in cu. ft.,

N = Number of bags of cement in the batch,

94 = Net weight of a bag of cement, in lbs.,

Wf = Total weight of fine aggregate in batch in condition used, in lbs.,

Wc = Total weight of coarse aggregate in batch in condition used, in lbs.,

Ww = Total weight of mixing water added to batch, in lbs., and

W = Weight of concrete, in lbs. per cu. ft.

Relative Yield:

$$Ry = \frac{Y}{Yd}$$

Y = Yield of concrete produced per batch, in yd^3,

Yd = Theoretical Yield (yd^3)

NOTE: A value for Ry greater than 1.00 indicates an excess of concrete being produced, while a value less than 1.00 indicates the batch to be "short" of its designed volume.

Where: Ry = Relative Yield of concrete

(a) Unit Weight

The net weight of the concrete shall be calculated by subtracting the weight of the measure used in the test from the gross weight. The unit weight shall be calculated by multiplying the net weight by the factor for the measure used. The method of determining this factor is given in AASHTO T121.

Modulus of Rupture:

= $(7.5\sqrt{f'c})$ or

$$\text{Modulus of Rupture, in psi} = \frac{3WL}{2bd^2}$$

Where: W = Maximum indicated load, in lbs.,

L = Distance between supports, in in., and

b & d = Breadth and depth of beam, in in.

With a 6" x 6" x 40" beam, this formula resolves to: $\frac{72W}{432} = \frac{W}{6}$

Therefore, 1/6 of the gage reading equals the modulus of rupture of the tested specimen, in lbs. per sq. in.