FE Exam Review

Civil Specific Section
Geotechnical Engineering
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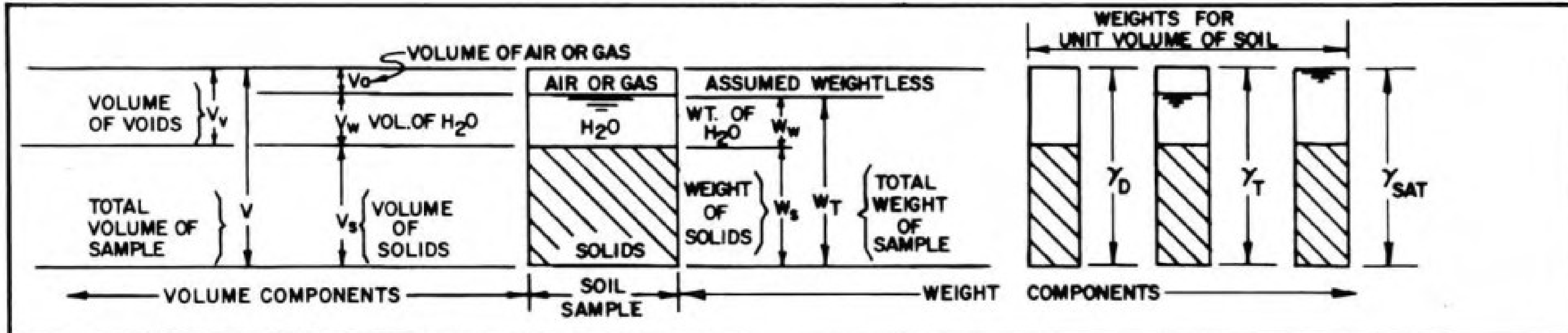
M. ASCE

TABLE 6
Volume and Weight Relationships



		PROPERTY	SATURATED SAMPLE (W_s, W_w, G, ARE KNOWN)	UNSATURATED SAMPLE (W_s, W_w, G, V, ARE KNOWN)	SUPPLEMENTARY FORMULAS RELATING MEASURED AND COMPUTED FACTORS				
VOLUME COMPONENTS	V_s	VOLUME OF SOLIDS	$\frac{W_s}{G\gamma_w}$		$V-(V_a+V_w)$	$V(1-n)$	$\frac{V}{(1+e)}$	$\frac{V_v}{e}$	
	V_w	VOLUME OF WATER	$\frac{W_w}{\gamma_w}$		V_v-V_a	SV_v	$\frac{SVe}{(1+e)}$	$SV_s e$	
	V_a	VOLUME OF AIR OR GAS	ZERO	$V-(V_s+V_w)$	V_v-V_w	$(1-S)V_v$	$\frac{(1-S)V_e}{(1+e)}$	$(1-S)V_s e$	
	V_v	VOLUME OF VOIDS	$\frac{W_w}{\gamma_w}$	$V-\frac{W_s}{G\gamma_w}$	$V-V_s$	$\frac{V_s n}{1-n}$	$\frac{Ve}{(1+e)}$	$V_s e$	
	V	TOTAL VOLUME OF SAMPLE	V_s+V_w	MEASURED	$V_s+V_a+V_w$	$\frac{V_s}{1-n}$	$V_s(1+e)$	$\frac{V_v(1+e)}{e}$	
	n	POROSITY	$\frac{V_v}{V}$		$1-\frac{V_s}{V}$	$1-\frac{W_s}{GV\gamma_w}$	$\frac{e}{1+e}$		
	e	VOID RATIO	$\frac{V_v}{V_s}$		$\frac{V}{V_s}-1$	$\frac{GV\gamma_w}{W_s}-1$	$\frac{W_w G}{W_s S}$	$\frac{n}{1-n}$	$\frac{wG}{S}$

TABLE 6 (continued)
Volume and Weight Relationships

		PROPERTY	SATURATED SAMPLE (W_s, W_w, G, ARE KNOWN)	UNSATURATED SAMPLE (W_s, W_w, G, V, ARE KNOWN)	SUPPLEMENTARY FORMULAS RELATING MEASURED AND COMPUTED FACTORS			
WEIGHTS FOR SPECIFIC SAMPLE	W_s	WEIGHT OF SOLIDS	MEASURED		$\frac{W_T}{(1+W)}$	$GV\gamma_w(1-n)$	$\frac{W_w G}{eS}$	
	W_w	WEIGHT OF WATER	MEASURED		wW_s	$S\gamma_w V_v$	$\frac{eW_s S}{G}$	
	W_t	TOTAL WEIGHT OF SAMPLE	$W_s + W_w$		$W_s(1+w)$			
WEIGHTS FOR SAMPLE OF UNIT VOLUME	γ_D	DRY UNIT WEIGHT	$\frac{W_s}{V_s + V_w}$	$\frac{W_s}{V}$	$\frac{W_t}{V(1+w)}$	$\frac{G\gamma_w}{(1+e)}$	$\frac{G\gamma_w}{1+wG/S}$	
	γ_T	WET UNIT WEIGHT	$\frac{W_s+W_w}{V_s+V_w}$	$\frac{W_s+W_w}{V}$	$\frac{W_T}{V}$	$\frac{(G+Se)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w/S+1/G}$	
	γ_{SAT}	SATURATED UNIT WEIGHT	$\frac{W_s+W_w}{V_s+V_w}$	$\frac{W_s+V_v\gamma_w}{V}$	$\frac{W_s}{V}+\left(\frac{e}{1+e}\right)\gamma_w$	$\frac{(G+e)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w+1/G}$	
	γ_{SUB}	SUBMERGED (BUOYANT) UNIT WEIGHT	$\gamma_{SAT} - \gamma_w$		$\frac{W_s}{V}-\left(\frac{1}{1+e}\right)\gamma_w$	$\left(\frac{G+e}{1+e}-1\right)\gamma_w$	$\left(\frac{1-1/G}{w+1/G}\right)\gamma_w$	
COMBINED RELATIONS	w	MOISTURE CONTENT	$\frac{W_w}{W_s}$		$\frac{W_t}{W_s}-1$	$\frac{Se}{G}$	$S\left[\frac{\gamma_w}{\gamma_D}-\frac{1}{G}\right]$	
	S	DEGREE OF SATURATION	1.00	$\frac{V_w}{V_v}$	$\frac{W_w}{V_v\gamma_w}$	$\frac{wG}{e}$	$\frac{w}{\left[\frac{\gamma_w}{\gamma_D}-\frac{1}{G}\right]}$	
	G	SPECIFIC GRAVITY	$\frac{W_s}{V_s\gamma_w}$		$\frac{Se}{w}$			

Relative Density

- Definition of Relative Density (I_D or D_r both used)

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}}$$

  - e_{max} = void ratio of the soil in its loosest condition
  - e_{min} = void ratio of the soil in its densest condition
  - e = void ratio in the natural or condition of interest of the soil
  - Convenient measure for the strength of a cohesionless soil

- Alternative definition:

$$D_r = \frac{\gamma_{d_{max}}\left(\gamma_{d_{field}} - \gamma_{d_{min}}\right)}{\gamma_{d_{field}}\left(\gamma_{d_{max}} - \gamma_{d_{min}}\right)} \times 100\%$$

  - γ_{dmax} = dry density in the densest condition
  - γ_{dmin} = dry density in the loosest condition
  - γ_{dfield} = dry density in the field

Relative Compaction

Due to the obvious disconnect between the types of energy in the laboratory and the field, some method is needed to express the laboratory-measured compaction parameters, i.e., maximum dry unit weight ($\gamma_{d\text{-}max}$) and optimum moisture content (w_{opt}), in terms of field compaction. Most commonly, this relationship is achieved by so-called **performance based or end-product specifications** wherein a certain **relative compaction, RC,** also known as **percent compaction**, is specified. The RC is simply the ratio of the desired field dry unit weight, $\gamma_{d\ field}$, to the maximum dry density measured in the laboratory, $\gamma_{d\ max}$, expressed in percent as follows:

$$RC = \frac{\gamma_{d\,field}}{\gamma_{d\,max}} \text{ x } 100\% \qquad 5\text{-}19$$

Coefficients of Uniformity and Curvature; Atterberg Limits

Gradation Characteristics:				
Effective diameter	D_{10}	L	From grain size curve.	
Percent grain size	D_{30} D_{60} D_{85}	L	From grain size curve.	Classification, estimating permeability and unit weight, filter design, grout selection, and evaluating potential frost heave and liquefaction.
Coefficient of uniformity	C_u	D	$\frac{D_{60}}{D_{10}}$	
Coefficient of curvature	C_z	D	$\frac{(D_{30})^2}{(D_{10})x(D_{60})}$	

Plasticity Characteristics:				
Liquid Limit	LL	D	Directly from test	Classification and properties correlation.
Plastic Limit	PL	D	Directly from test	
Plasticity Index	PI	D	LL-PL	

Permeability

4. PERMEABILITY. Field permeability tests measure the coefficient of permeability (hydraulic conductivity) of in-place materials. The coefficient of permeability is the factor of proportionality relating the rate of fluid discharge per unit of cross-sectional area to the hydraulic gradient (the pressure or "head" inducing flow, divided by the length of the flow path). This relation is usually expressed simply:

$$Q/A = \frac{HK}{L}$$

Where Q is discharge (volume/time); A is cross-sectional area, $\frac{H}{L}$ is the hydraulic gradient (dimensionless); and K is the coefficient of permeability, expressed in length per unit time (cm/sec, ft/day, etc.). The area and length factors are often combined in a "shape factor" or "conductivity coefficient." See Figure 13 for analysis of observations and Table 15 for methods of computation. Permeability is the most variable of all the material properties commonly used in geotechnical analysis. A permeability spread of ten or more orders of magnitude has been reported for a number of different types of tests and materials. Measurement of permeability is highly sensitive to both natural and test conditions. The difficulties inherent in field permeability testing require that great care be taken to minimize sources of error and to correctly interpret, and compensate for, deviations from ideal test conditions.

Constant Head Permeability Test

- Compute the coefficient of permeability, k for laboratory test:
  - k_{20} = coefficient of permeability, cm/sec at 20° C
  - Q= quantity of flow, cm^3
  - L = length of specimen over which head loss is measured, cm.
  - h = loss of head in length, L, or difference in piezometer readings = h_1 - h_2, cm
  - A = cross-sectional area of specimen, cm^2
  - t = elapsed time, sec.

$$v=ki$$

$$\frac{Q}{At}=k\frac{\Delta h}{\Delta L}$$

Solving,

$$k=\frac{Q\Delta L}{\Delta hAt}$$

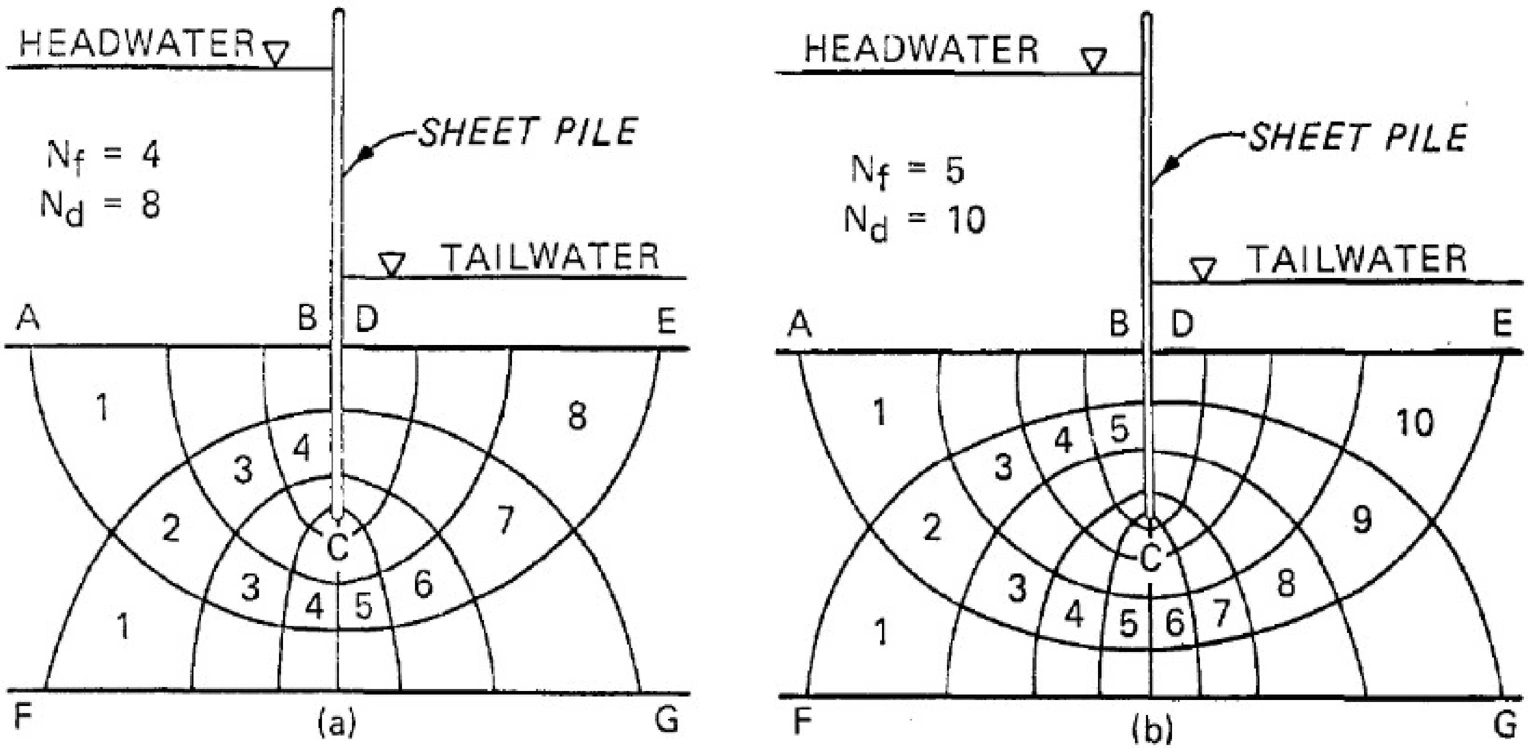
Falling Head Permeability Test

- Compute the coefficient of permeability, k

$$k = \frac{aL}{At}\left(\ln \frac{h_o}{h_f}\right)$$

  - a = inside area of standpipe, cm^2
  - A = cross-sectional area of specimen, cm^2
  - L = length of specimen, cm
  - t = elapsed time (t_f - t_o), sec
  - h_o = height of water in standpipe above discharge level at time t_o, cm
  - h_f = height of water in standpipe above discharge level at time t_f, cm

Flow Nets-Counting



a. Net drawn for four flow channels.

b. Net drawn for five flow channels.

Figure 4-5. Flow net for a sheetpile wall in a permeable foundation (from U. S. Army Engineer District, Little Rock[92])

Flow Nets—Interpretation

- Basic Equation

$$q = kh_w \left(\frac{N_f}{N_d} \right)$$

  - h_w = elevation difference between upstream and downstream limits of water head driving the flow net
  - N_f = number of flow channels for net
  - N_d = number of equipotential drops for net
  - Assumes that the "aspect ratio" of the flow spaces is unity

Soil Boiling and Critical Gradient

Critical Gradient:

$$i_c = \frac{\gamma_s}{\gamma_w} - 1 = \frac{\gamma_{sub}}{\gamma_w}$$

Exit Gradient i_e is the gradient where the water flowing through a net exits

Factor of safety against seepage:

$$FS = \frac{i_c}{i}$$

Primary Consolidation Settlement

Ultimate consolidation settlement in soil layer:

$$S_{ULT}=\epsilon_v H_s$$

$$\epsilon_v=\frac{\Delta e_{TOT}}{1+e_0}=\frac{\Delta H}{H_s}$$

Stress increase:

$$\Delta p=Iq_s$$

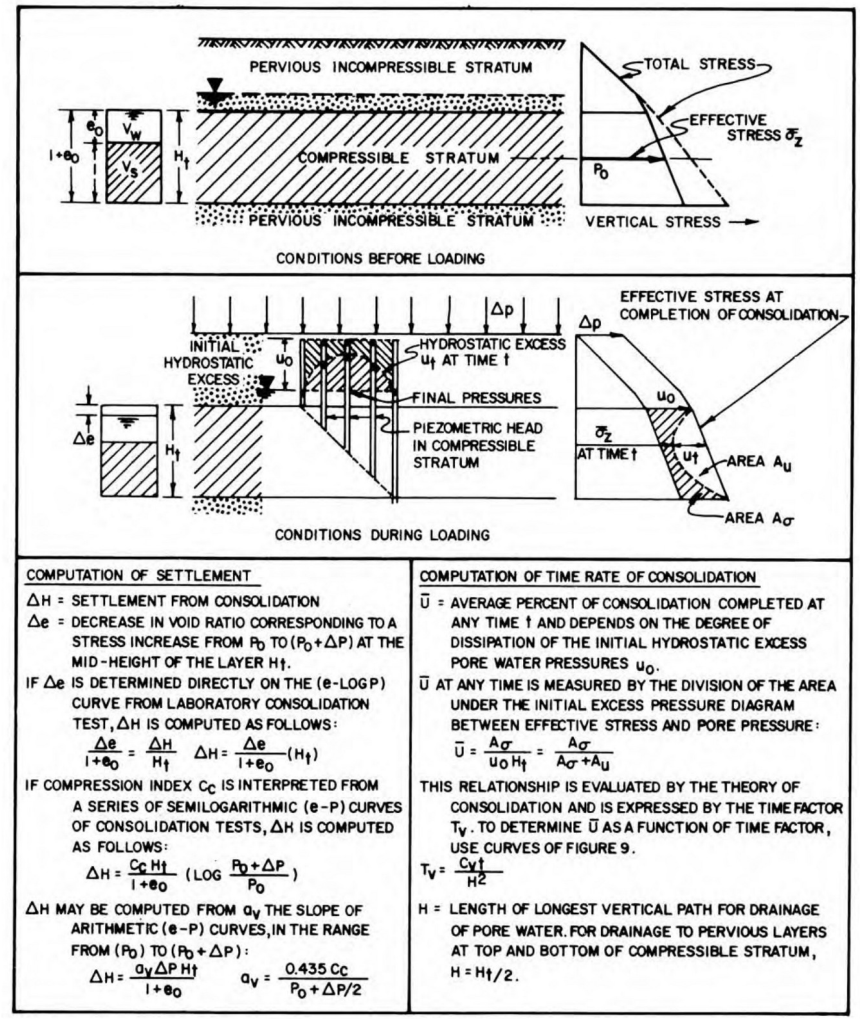


FIGURE 1
Consolidation Settlement Analysis

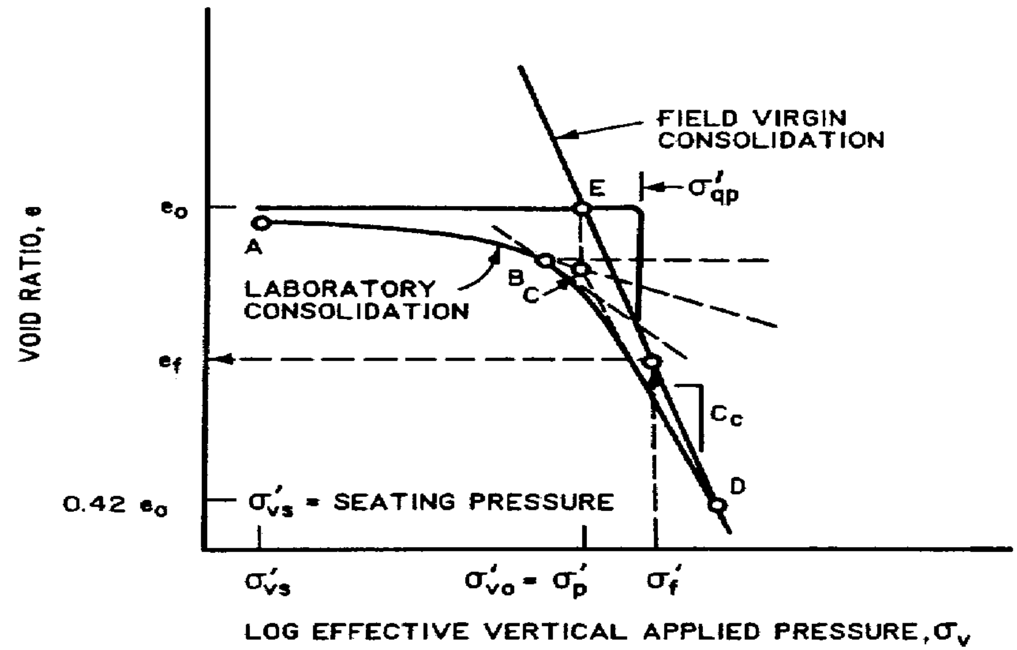
Normally Consolidated Soils

Compression Index

$$C_c = \frac{\Delta e}{\Delta \log(p)}$$

As a function of LL:

$$C_c = 0.009(\mathrm{LL} - 10)$$



$$S_c = \sum_i^n \frac{C_c}{1+e_o} H_o \log_{10}\left(\frac{p_f}{p_o}\right) \qquad 7\text{-}2$$

where:

C_c	=	compression index
e_o	=	initial void ratio
H_o	=	layer thickness
p_o	=	initial effective vertical stress at the center of layer n
p_f	=	$p_o + \Delta p$ = final effective vertical stress at the center of layer n.

Preconsolidated or Overconsolidated Soils

- Formula, $\sigma'_o + \Delta\sigma' > \sigma'_c$ (to the right of "F")

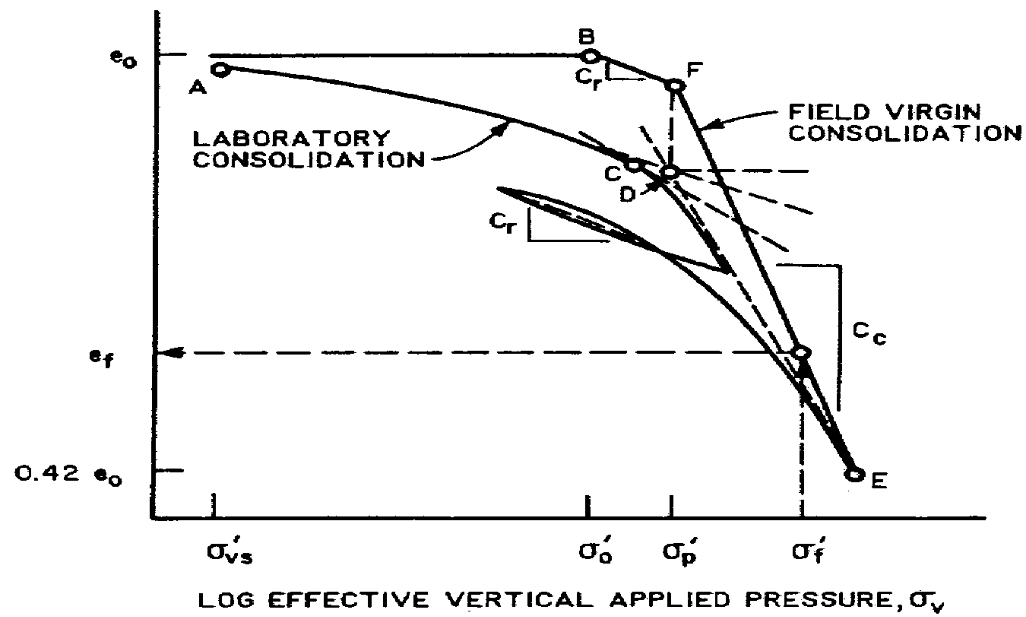
$$S_c = \frac{C_s H}{1+e_o}\log\left(\frac{\sigma'_c}{\sigma'_o}\right) + \frac{C_c H}{1+e_o}\log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_c}\right)$$

- Formula, $\sigma'_o + \Delta\sigma' < \sigma'_c$ (to the left of "F")

$$S_c = \frac{C_s H}{1+e_o}\log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

- Recompression Index

$$C_r = \frac{\Delta e}{\Delta \log(p)} = \frac{C_c}{6}$$



Consolidation: Settlement in Time

- Time Factor

$$T_v = \frac{tc_v}{H_{dr}^2}$$

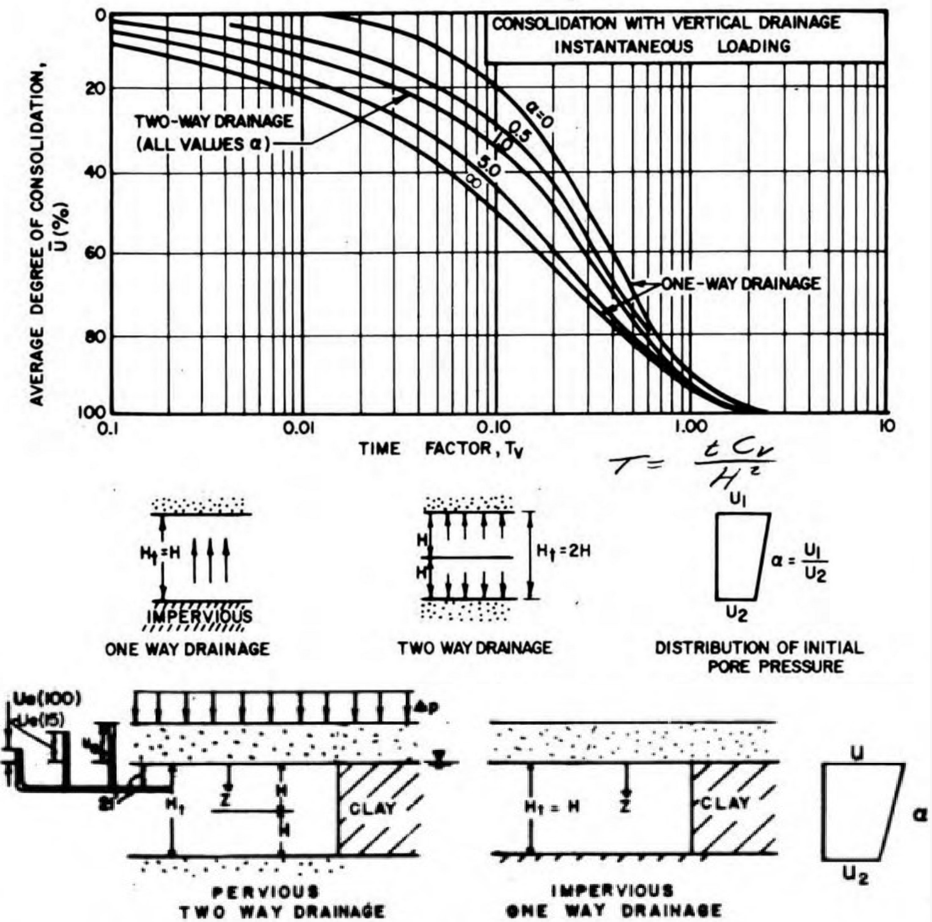
  - t = time of consolidation
  - T_v = time factor for vertical drainage
  - H_{dr} = longest distance of drainage to pervious surface
    - Usually depth of layer for single drainage, half of layer depth for double drainage
  - c_v = coefficient of consolidation

- Degree of Consolidation

$$U = \frac{S_{c(t)}}{S_c} = 1 - \frac{u_e}{u_o}$$

  - Expressed as a percentage
  - Expresses ratio of both pore pressures and settlement
  - Allows relating settlement to time (consolidation)

Consolidation: Settlement in Time



$U_z = 1 - \frac{u_e}{u_o}$ (consolidation ratio)

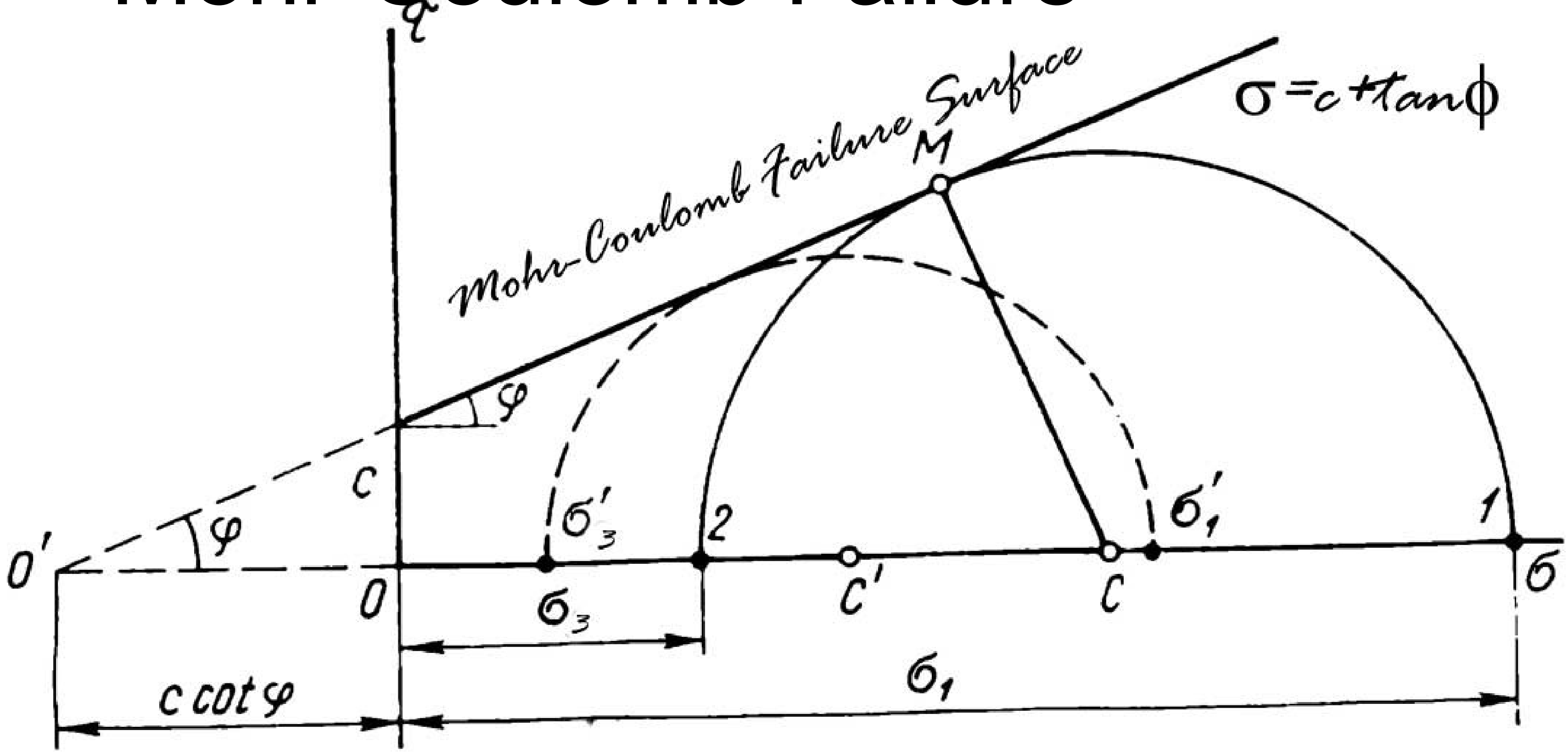
u_e = Excess pore pressure at some time t

u_o = Excess pore pressure at time t = 0 (due to external loading)

Table 7-4
Average degree of consolidation, U, versus Time Factor, T_v, for uniform initial increase in pore water pressure

U %	T_v
0	0.000
10	0.008
20	0.031
30	0.071
40	0.126
50	0.197
60	0.287
70	0.403
80	0.567
90	0.848
93.1	1.000
95.0	1.163
98.0	1.500
99.4	2.000
100.0	Infinity

Mohr-Coulomb Failure



s = mean normal stress = $(\sigma_1+\sigma_3)/2$

σ_1 = major principal stress

σ_3 = minor principal stress

c = cohesion

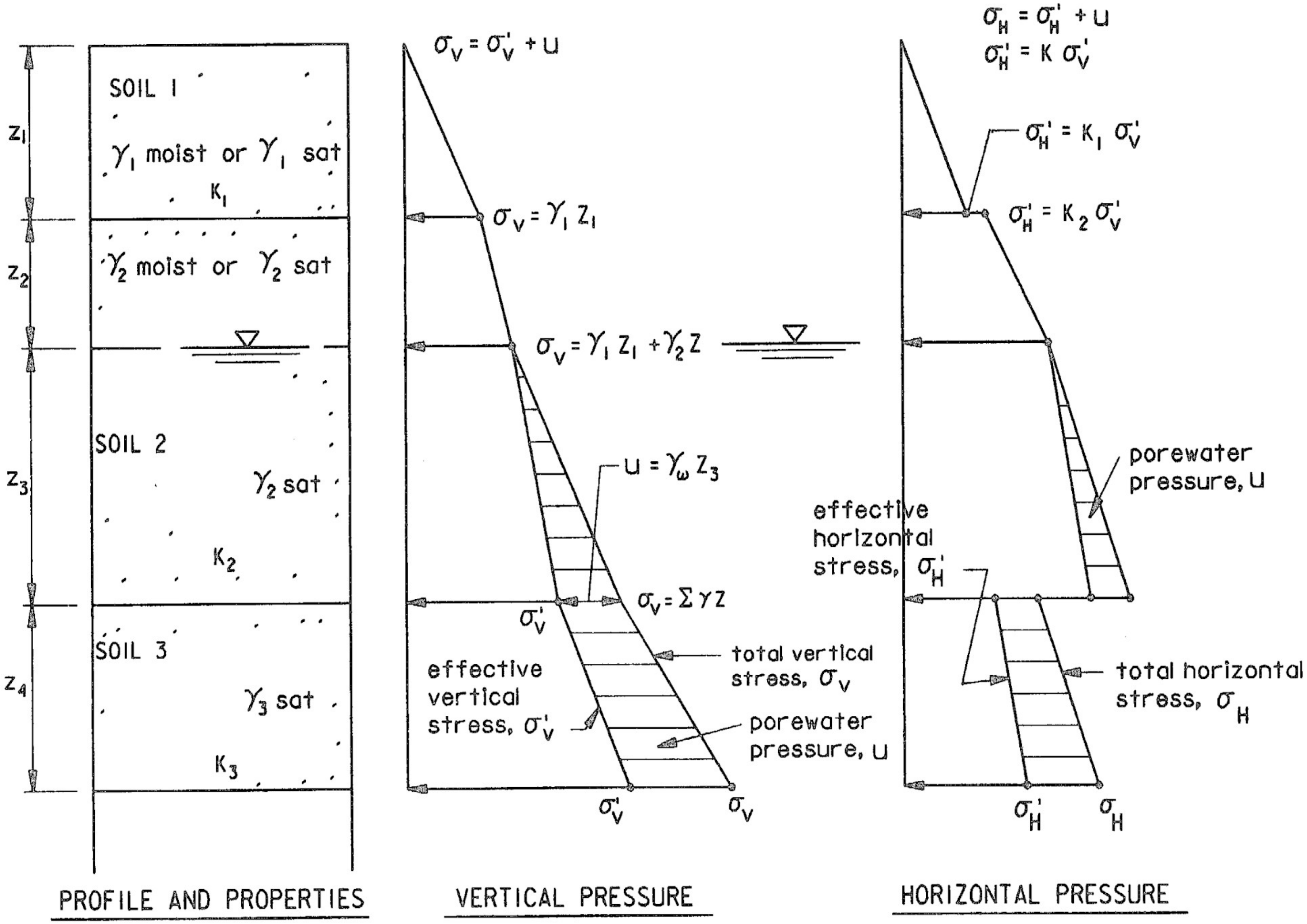
φ = friction angle

Stresses in Soils

- σ_N = total normal stress = P/A
  - P = normal force
  - A = cross-sectional area over which force acts
- τ = shear stress = T/A
  - T = shearing force
- σ' = effective stress = $\sigma - u$
  - u = pore water pressure = $h_u \gamma_w$
  - h_u = uplift or pressure head

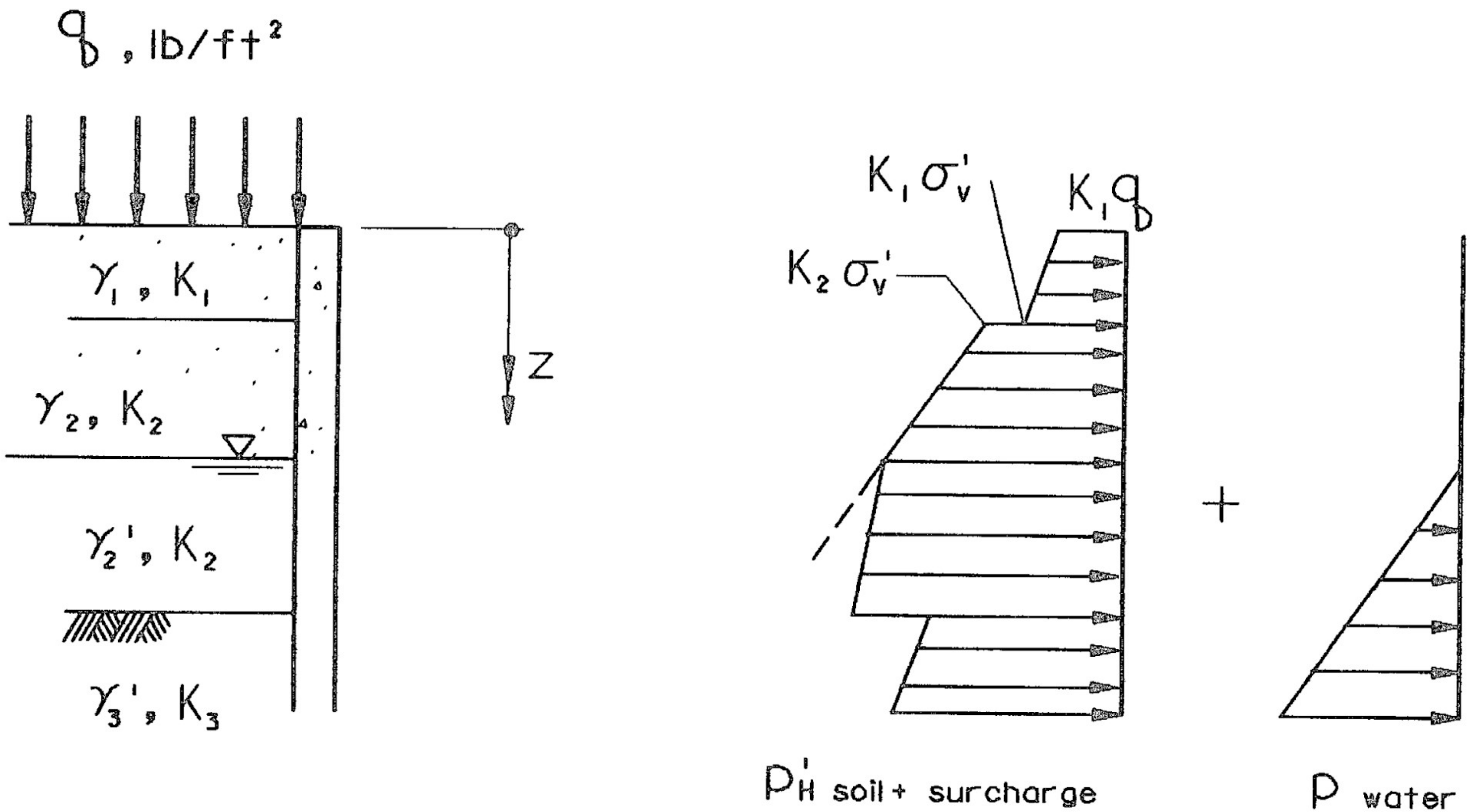
Vertical and Horizontal Pressures in Soils

From EM 1110-2-2502

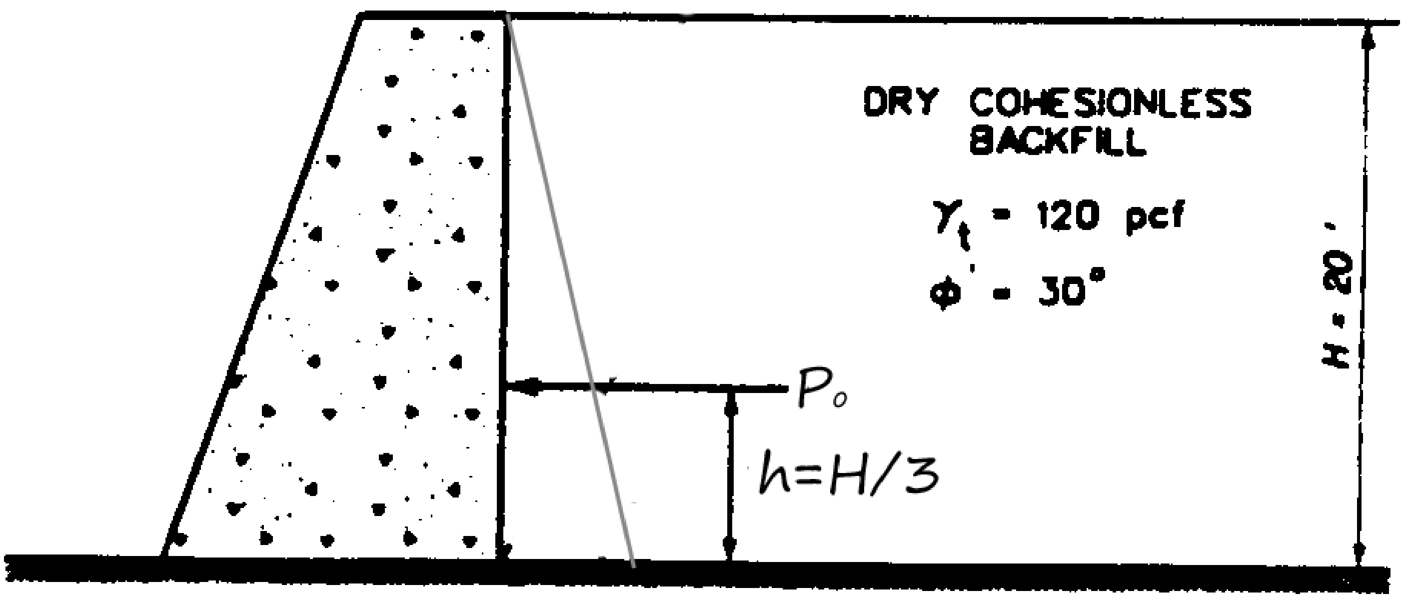


Uniform Surcharge and Unbalanced Hydrostatic Forces

From EM 1110-2-2502



At Rest Earth Pressures



$$K_o = 1 - \sin\varphi$$

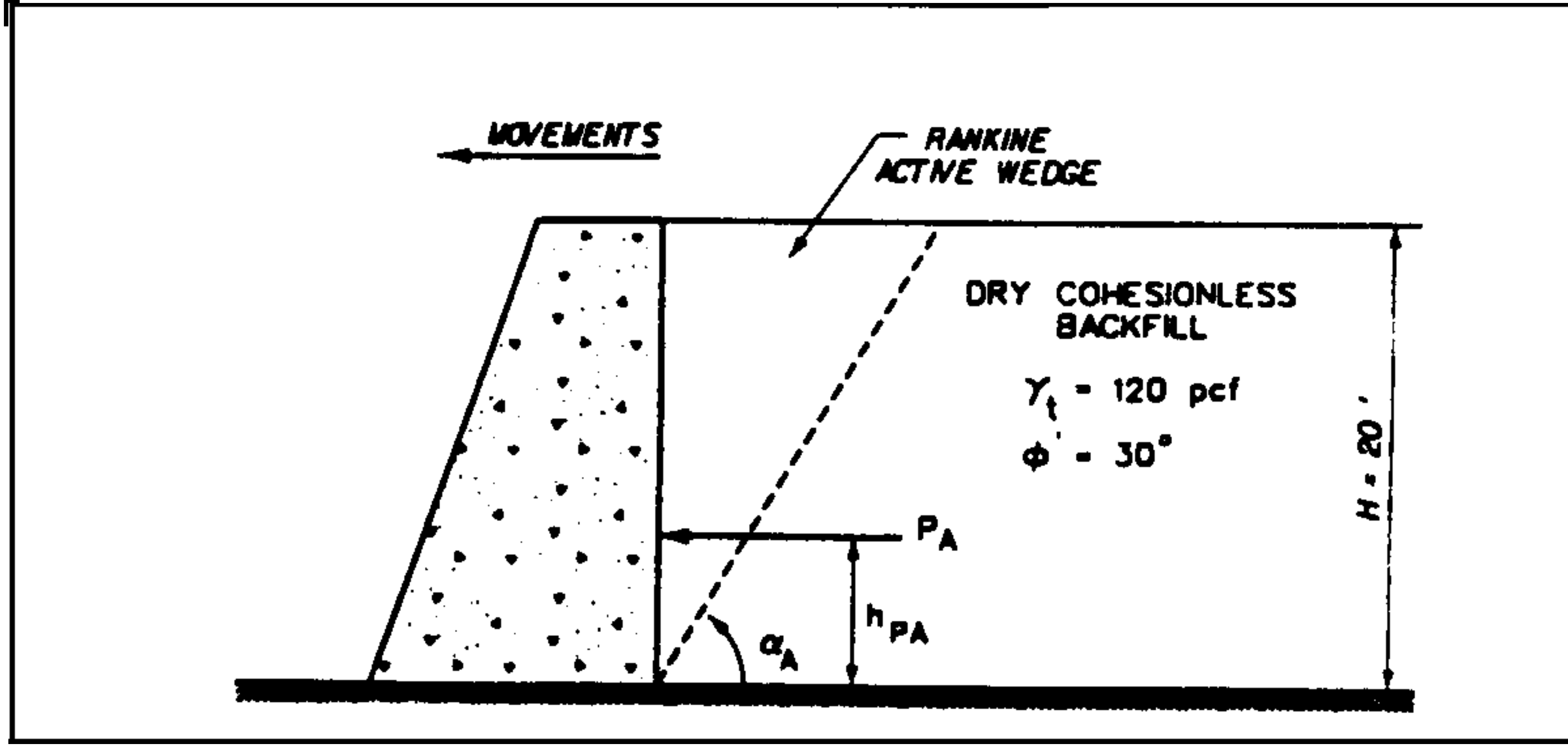
Normally Consolidated

$$K_o = 1 - \sin\varphi (OCR)^{\sin\varphi}$$

Overconsolidated or Preconsolidated

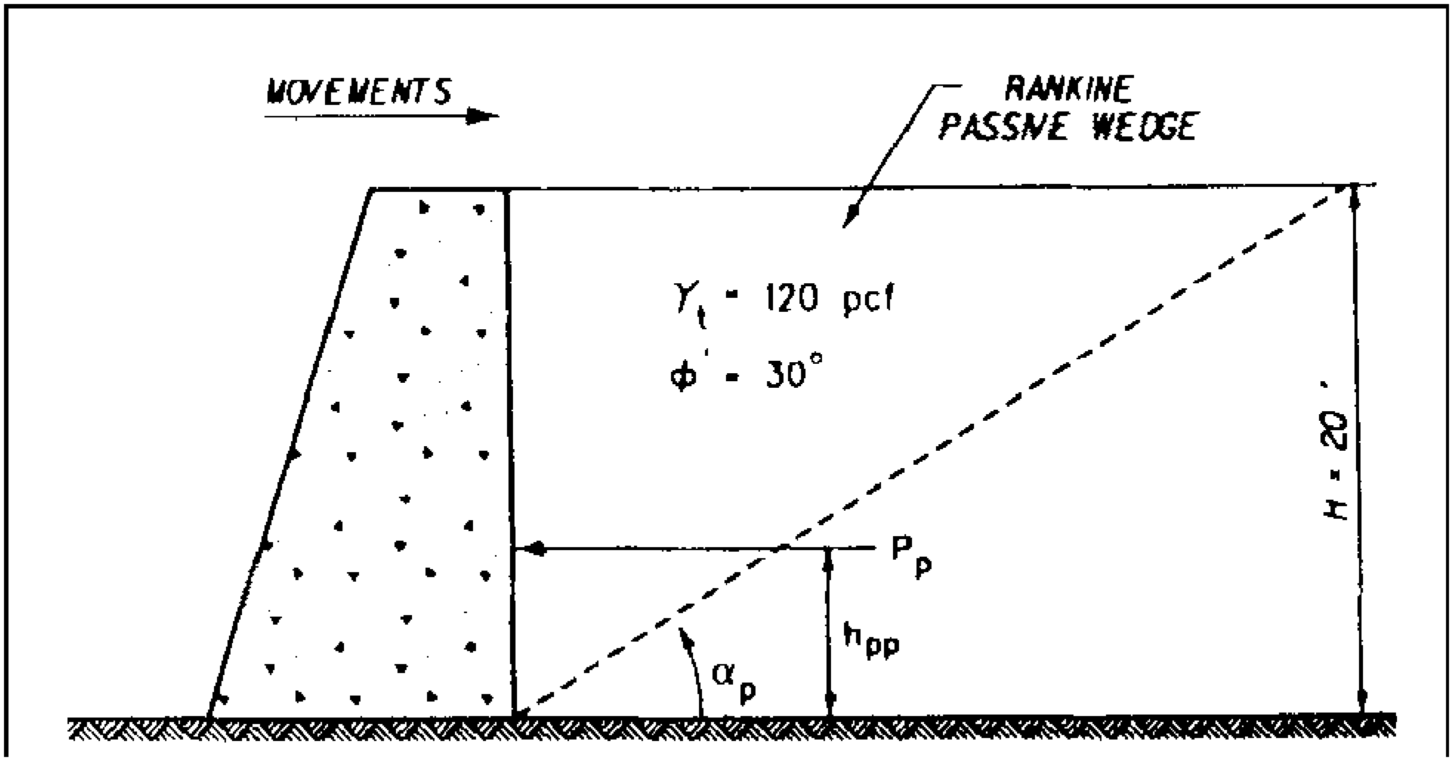
$$P = \frac{\gamma H^2 K_o}{2}$$

Rankine Active Earth Pressures



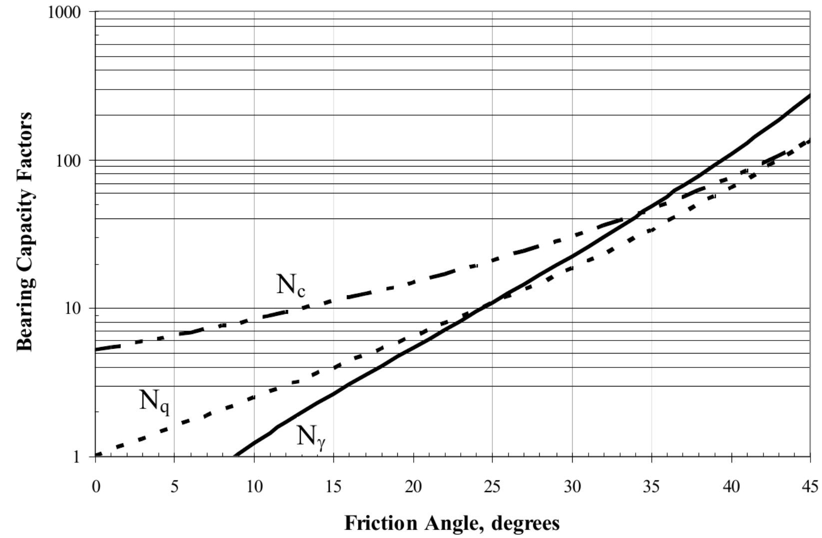
$$K_a = tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right)^2 ; P = \frac{\gamma H^2 K_a}{2}$$

Rankine Passive Earth Pressures



$$K_p = tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right)^2 ; P = \frac{\gamma H^2 K_p}{2}$$

Bearing Capacity



$$q_{ult} = \underbrace{c\,(N_c)}_{\text{“Cohesion” term}} + \underbrace{q\,(N_q)}_{\text{“Surcharge” term}} + \underbrace{0.5\,(\gamma)(B_f)(N_\gamma)}_{\text{Foundation soil “Weight” term}}$$

where: c = cohesion of the soil (ksf) (kPa)

q = total surcharge at the base of the footing = $q_{appl} + \gamma_a D_f$ (ksf) (kPa)

- q_{appl} = applied surcharge (ksf)(kPa)
- γ_a = unit weight of the overburden material above the base of the footing causing the surcharge pressure (kcf) (kN/m^3)

D_f = depth of embedment (ft) (m) (Figure 8-1)

γ = unit weight of the soil under the footing (kcf) (kN/m^3)

B_f = footing width, i.e., least lateral dimension of the footing (ft) (m) (Figure 8-1)

N_q = bearing capacity factor for the “surcharge” term (dimensionless)

$$= e^{\pi \tan\phi} \tan^2 (45° + \frac{\phi}{2}) \qquad 8\text{-}2$$

N_c = bearing capacity factor for the “cohesion” term (dimensionless)

$$= (N_q - 1) \cot\phi \qquad \text{for } \phi > 0° \qquad 8\text{-}3$$

$$= 2 + \pi = 5.14 \qquad \text{for } \phi = 0° \qquad 8\text{-}4$$

N_γ = bearing capacity factor for the “weight” term (dimensionless)

$$= 2\,(N_q + 1) \tan(\phi) \qquad 8\text{-}5$$

Retaining Walls

Notes:

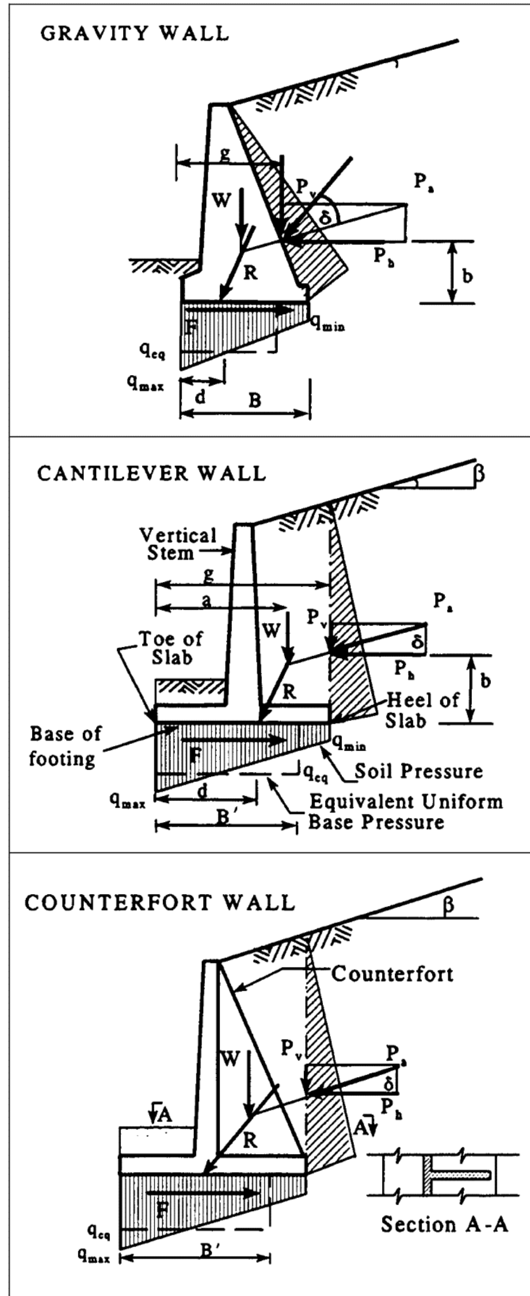
Sliding FS contains additional term in denominator for passive pressure

Maximum base stress q_{max} is referred to as q_{toe}, which in fact it is

$\delta = k_1\varphi_1$, $c_a = k_2C_2$

$\frac{1}{2} \leq k_1$ or $k_2 \leq 2/3$, given

$$FS_{overturning} = \frac{M_{resisting}}{M_{driving}} = \frac{\sum M_R}{\sum M_0}$$



DEFINITIONS

B = width of the base of the footing

$\tan\delta_b$ = friction factor between soil and base (see Table 10-1 for guidance)

W = weight at the base of wall. Includes weight of wall for gravity walls. Includes weight of the soil above footing for cantilever and counterfort walls

c = cohesion of the foundation soil

c_a = adhesion between concrete and soil

δ = angle of wall friction

P_p = passive resistance

LOCATION OF RESULTANT, R

Based on moments about toe (assuming P_p=0)

$$d = \frac{Wa + P_v g - P_h b}{W + P_v}$$

CRITERIA FOR ECCENTRICITY, e

$$e = d - \frac{B}{2}$$; $e \leq B/6$ for soils; $e \leq B/4$ for rocks

FACTOR OF SAFETY AGAINST SLIDING

$$FS_s = \frac{(W + P_v)\tan\delta_b + c_a B}{P_h} \geq 1.5\,(\text{min})$$

APPLIED STRESS AT BASE (q_{max}, q_{min}, q_{eq})

$$\boxed{q_{max} = \frac{(W + P_v)}{B}\left(1 + \frac{6e}{B}\right)}$$

$$q_{min} = \frac{(W + P_v)}{B}\left(1 - \frac{6e}{B}\right)$$

Equivalent uniform (Meyerhof) applied stress, q_{eq}, is given as follows:

$$q_{eq} = \frac{(W + P_v)}{B'} \text{ where } B' = B - 2e$$

Use uniform stress, q_{eq}, for soils and settlement analysis; use trapezoidal distribution with q_{max} and q_{min} for rocks and structural analysis

DEEP-SEATED (GLOBAL) STABILITY

Evaluate global stability using guidance in Chapter 6 (Slope Stability)

Figure 10-17. Design criteria for cast-in-place (CIP) Concrete retaining walls (after NAVFAC, 1986b).

Planar Sliding Slope Stability

$T = w_m \sin \beta$ (driving force)

$N = w_m \cos \beta$ (normal force)

$S = cL_s + N \tan \varphi$ (resisting force)

$FS = S/T$

w_m = weight of soil mass

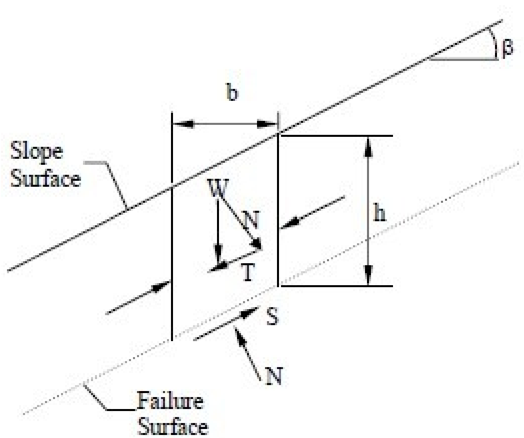
β = slope angle

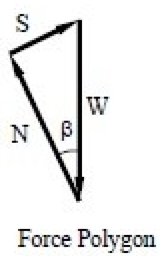
c = soil cohesion

L_s = contact of soil mass with slope

φ = friction angle of soil

FS = factor of safety





Unified Soil Classification

Table 4-9

Soil classification chart (laboratory method) (after ASTM D 2487)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[a]			Soil Classification: Group Symbol	Soil Classification: Group Name[b]
COARSE-GRAINED SOILS (Sands and Gravels) - more than 50% retained on No. 200 (0.075 mm) sieve **FINE-GRAINED (Silts and Clays)** - 50% or more passes the No. 200 (0.075 mm) sieve				
GRAVELS More than 50% of coarse Fraction retained on No. 4 Sieve	CLEAN GRAVELS < 5% fines	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW	Well-graded gravel[f]
		$C_u < 4$ and/or $1 > C_c > 3$[e]	GP	Poorly-graded gravel[f]
	GRAVELS WITH FINES > 12% of fines[c]	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
		Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
SANDS 50% or more of coarse fraction passes No. 4 Sieve	CLEAN SANDS < 5% fines[d]	$C_u > 6$ and $1 < C_c < 3$[c]	SW	Well-graded Sand[i]
		$C_u < 6$ and/or $1 > C_c > 3$[e]	SP	Poorly-graded sand[i]
	SANDS WITH FINES > 12% fines[d]	Fines classify as ML or MH	SM	Silty sand[g,h,i]
		Fines classify as CL or CH	SC	Clayey sand[g,h,i]
SILTS AND CLAYS Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line[j]	CL	Lean clay[k,l,m]
		PI < 4 or plots below "A" line[j]	ML	Silt[k,l,m]
	Organic	$\frac{\text{Liquid limit - overdried}}{\text{Liquid limit - not dried}} < 0.75$	OL	Organic clay[k,l,m,n]
				Organic silt[k,l,m,o]
SILTS AND CLAYS Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[k,l,m]
		PI plots below "A" line	MH	Elastic silt[k,l,m]
	Organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OH	Organic clay[k,l,m,p]
				Organic silt[k,l,m,q]
Highly fibrous organic soils	Primary organic matter, dark in color, and organic odor		Pt	Peat

Table 4-9 (Continued)

Soil classification chart (laboratory method) (after ASTM D 2487)

NOTES:

a Based on the material passing the 3 in (75 mm) sieve.

b If field sample contained cobbles and/or boulders, add "with cobbles and/or boulders" to group name.

c Gravels with 5 to 12% fines require dual symbols:
- GW-GM, well-graded gravel with silt
- GW-GC, well-graded gravel with clay
- GP-GM, poorly graded gravel with silt
- GP-GC, poorly graded gravel with clay

d Sands with 5 to 12% fines require dual symbols:
- SW-SM, well-graded sand with silt
- SW-SC, well-graded sand with clay
- SP-SM, poorly graded sand with silt
- SP-SC, poorly graded sand with clay

e $C_u = \frac{D_{60}}{D_{10}} \quad C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})}$

[C_u: Uniformity Coefficient; C_c: Coefficient of Curvature]

f If soil contains ≥ 15% sand, add "with sand" to group name.

g If fines classify as CL-ML, use dual symbol GC-GM, SC-SM.

h If fines are organic, add "with organic fines" to group name.

i If soil contains ≥ 15% gravel, add "with gravel" to group name.

j If the liquid limit and plasticity index plot in hatched area on plasticity chart, soil is a CL-ML, silty clay.

k If soil contains 15 to 29% plus No. 200 (0.075 mm), add "with sand" or "with gravel," whichever is predominant.

l If soil contains ≥ 30% plus No. 200 (0.075mm), predominantly sand, add "sandy" to group name.

m If soil contains ≥ 30% plus No. 200 (0.075 mm), predominantly gravel, add "gravelly" to group name.

n PI ≥ 4 and plots on or above "A" line.

o PI < 4 or plots below "A" line.

p PI plots on or above "A" line.

q PI plots below "A" line.

Unified Soil Classification

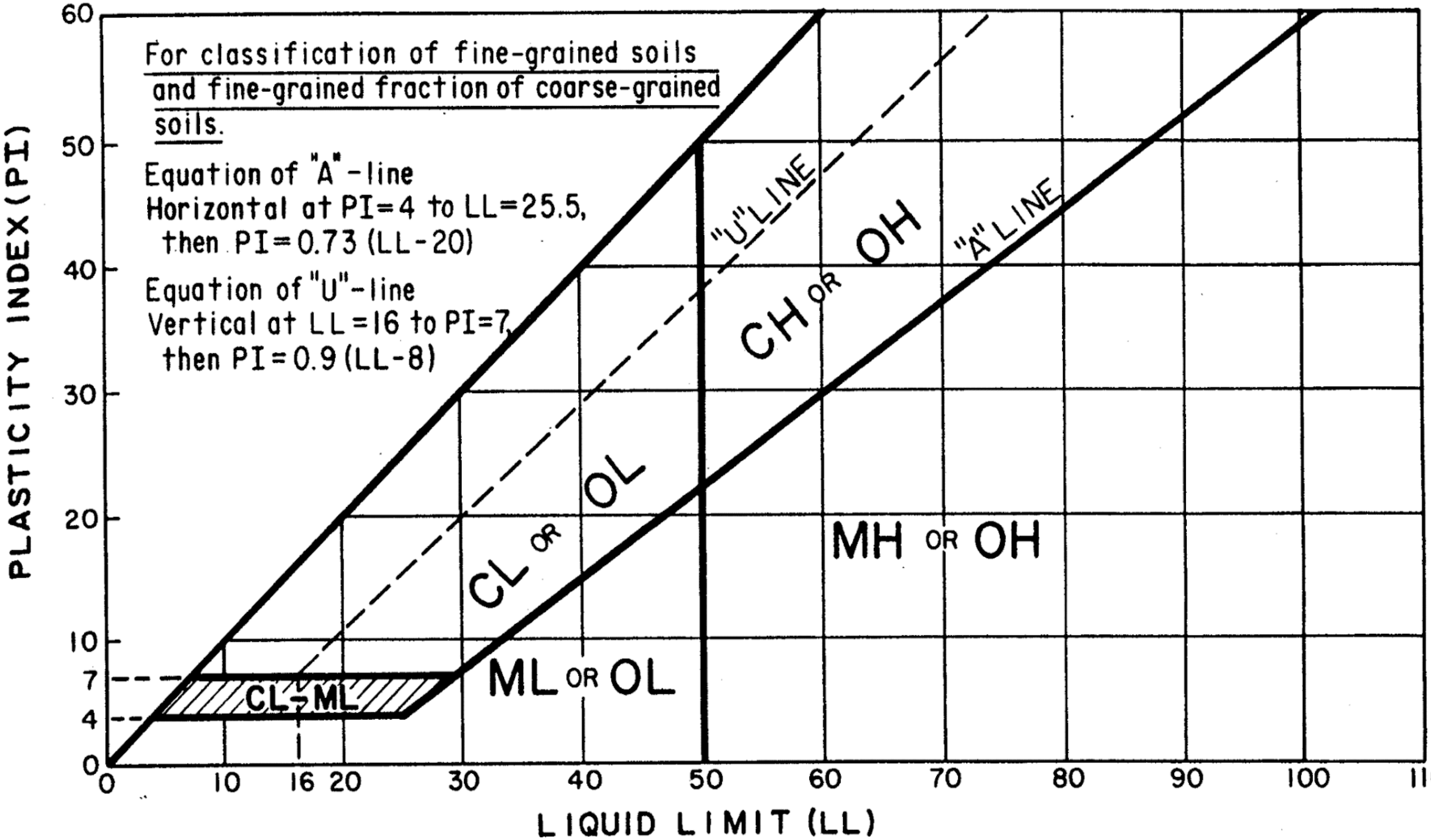


Figure 4-3. Plasticity chart for Unified Soil Classification System (ASTM D 2487).

AASHTO System

Table 4-13
AASHTO soil classification system based on AASHTO M 145 (or ASTM D 3282)

GENERAL CLASSIFICATION	**GRANULAR MATERIALS (35 percent or less of total sample passing No. 200 sieve (0.075 mm)**							**SILT-CLAY MATERIALS (More than 35 percent of total sample passing No. 200 sieve (0.075 mm)**			
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 (2 mm)	50 max.										
No. 40 (0.425 mm)	30 max.	50 max.	51 min.								
No. 200 (0.075 mm)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No 40 (0.425 mm)											
Liquid limit				40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
Group Index**	0		0	0		4 max.		8 max.	12 max.	16 max.	20 max.

Classification procedure:
With required test data available, proceed from left to right on chart; correct group will be found by process of elimination. The first group from left into which the test data will fit is the correct classification.

*Plasticity Index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity Index of A-7-6 subgroup is greater than LL minus 30 (see Fig 4-5).

**See group index formula (Eq. 4-3). Group index should be shown in parentheses after group symbol as: A-2-6(3), A-4(5), A-7-5(17), etc.

AASHTO System

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10)$$

where: F = Percent passing No. 200 sieve, expressed as a whole number. This percentage is based only on the material passing the No. 200 sieve.

LL = Liquid limit

PI = Plasticity index

The first term of Equation 4-3 is the partial group index determined from the liquid limit. The second term is the partial group index determined from the plasticity index. Following are some rules for determining group index:

- If Equation 4-3 yields a negative value for GI, it is taken as zero.
- The group index calculated from Equation 4-3 is rounded off to the nearest whole number, e.g., GI=3.4 is rounded off to 3; GI=3.5 is rounded off to 4.
- There is no upper limit for the group index.
- The group index of soils belonging to groups A-1-a, A-1-b, A-2-4, A-2-5, and A-3 will always be zero.
- When the group index for soils belonging to groups A-2-6 and A-2-7 is calculated, the partial group index for PI should be used, or

$$GI=0.01(F-15)(PI-10) \qquad 4\text{-}4$$

In general, the quality of performance of a soil as a subgrade material is inversely proportional to the group index.

Questions?