GEOTECHNICAL AND FOUNDATION FORMULA SHEET

Table Contents

Page

IDENTIFICATION AND CLASSIFICATION OF SOIL AND ROCK

1. The Coefficient of uniformity, $C_u = D_{60} / D_{10}$
2. The Coefficient of Curvature, $C_z = (D_{30})^2 / (D_{60} \times D_{10})$
3. Plasticity index, $PI = LL - PL$
4. Liquidity index, $LI = (w\text{-}PL) / (LL\text{-}PL)$
5. Activity index, $AI = PI / (\% > 0.002mm)$, Clay contain greater than 40%
6. Activity index, $AI = PI / (\% > 0.002mm - 5)$, Clay contain less than 40%
   if $AI => .75$, low active clay;
   if $AI = .75$ to 1.25, normal active clay ;
   if < 1.25, active clay
7. Group index, $GI = (F-35) \times [0.2 + 0.005 \times (LL-40)] + 0.01 \times (F-15) \times (PI-10)$,
   F IS % OF PASSING #200
8. VOLUME OF VOID, $V_v = V - V_s$;
9. VOLUME OF SOLID SOIL, $V_s = W_s / G_s \gamma_w$
10. VOLUME OF SOIL, $V = V_s + V_v$
11. TOTAL WEIGHT, $W = W_w + W_s$
12. WEIGHT of SOIL, $Ws = W / (1+w)$
13. WATER CONTENT, $w = W_w / W_s$
14. BULK DENSITY, $\gamma = W/V = G_s (1+w) \gamma_w / (1+e)$
    $= (G_s + S_r e) \gamma_w / (1+e)$
15. SATURATED UNIT WEIGHT, $\gamma_{sat} = (G_s + e) \gamma_w / 1+e$; $S_r = 1$
16. DRY UNIT WEIGHT, $\gamma_d = W_w / V = G_s \gamma_w / (1+e) = \gamma/(1+w)$
17. UNIT WEIGHT OF WATER, $\gamma_w = 62.4$ PCF $= 9.8 KN/m^3$
18. SUBMERGED UNIT WEIGHT, $\gamma' = (G_s - 1) \gamma_w / 1+e = \gamma_{sat} - \gamma_w$
    $= (G_s + e) \gamma_w / 1+e$
19. DEGREE OF SATURATION, $S_r = V_w / V_v = w G_s / e$
20. SPECIFIC GRAVITY, $G_s = W_s / V_s \gamma_w$
21. VOID RATIO, $e = V_v / V_s = n / 1-n = [G_s (1+w) \gamma_w / \gamma] - 1$
22. VOID RATIO, $e = w G_s / S_r$; WHERE FULLY SATURATED
23. SOIL, $S_r = 1$ POROSITY, $n = V_v / V$
24. SPECIFIC VOLUME, $v = 1 + e$
25. AIR CONTENT, $A = V_a / V = (e - w G_s) / 1+e = n (1 - S_r)$

26. RELATIVE DENSITY, $D_r = 100 (e_{max} - e) / (e_{max} - e_{min})$
    $D_r = 100(1/\gamma_{min} - 1/\gamma_d) / (1/\gamma_{min} - 1/\gamma_{max})$

27. Critical Hydraulic gradient, $i_c = \gamma'/\gamma_w = (G_s-1)/(1+e)$, Where, $\sigma'=0$

28. Terminal velocity of particle, $v = (\gamma - \gamma_w)D^2/18\mu_s$,
D=dia, μ_s=viscosity=.001 (SI unit)

HYDRAULIC PROPERTIES OF SOIL AND ROCK

29. DISCHARGE VELOCITY, $q = VA = kiA$, discharge,

$v = ki$; k = Coefficient of permeability
$i = \Delta h/L$ head loss over length of flow path
$V = ki = q/A = q/Ta = Q/At$,

30. VOLUME OF WATER, $Q = kiAt = k\,At\,\Delta h/L$

Q = Volume of water collected
k = Coefficient of permeability
i = Hydraulic gradient, h/L
A = Cross-sectional area of sample
t = Duration of time for collection of water
L = Length of the sample

For granular soil,

31. $K = 1/e^2$ For Horizontal flow

32. $K = e^3/1+e$ For vertical flow

33. Constant Head Permeability,

$k = QL/A\,\Delta ht$

34. Falling Head Permeability,

$k = 2.303(aL/At)\,\text{Log}_{10}(h_o/h_1)$
a = cross-sectional area of standpipe
h_o = water level in the standpipe at start of the time
h_1 = water level in the standpipe at end of the time

35. Equivalent Permeability of Stratified Deposit.

Equivalent Horizontal Permeability,
$K_{h(eq)} = (k_{h1} \times h_1 \quad + \quad k_{h2} \times h_2 \quad ..... \quad k_{hn} \times h_n \quad)$

36. Equivalent Horizontal Permeability,

$$K_{v(eq)} = \frac{h \quad . \qquad . \; .}{(h_1/k_{v1}) + (h_2/k_{v2}) .. .. \quad (h_n / k_{vn})}$$

37. DUPIT FORMULA FOR TWO DIMENSIONAL FLOWS ON A horizontal impervious boundary,

$$Q = k(h_1^2 - h_2^2) / 2L$$

38. Empirical coefficient of Permeability,

$k = CD_{10}2$, $C = .4$ to 1.5 , normally 1.0

$C_u < 5.0$

Confined Aquifer

39. Fully Penetrating
Coefficient of Permeability,

$$k = [2.303\ q\ Log_{10}(r_1 / r_2)]\ /\ 2\pi D(h_1 / h_2),$$

40. Partially Penetrating
Coefficient of Permeability,

$$k = [2.303\ q\ Log_{10}(r_1 / r_2)]\ /\ 2\pi D(h_1 / h_2)G,$$

$G = W / D\ [(1 + 7\ \sqrt{(r_w/2W)}\ \cos(\pi W/2D)]$

W = Partially Penetrating depth

r_w = Radius of the well

D = depth of aquifer

Unconfined Aquifer

41. Fully Penetrating
Coefficient of Permeability,

$$k = [2.303\ q\ Log_{10}(r_1 / r_2)]\ /\ \pi(h_2^2 - h_1^2)$$

41 Partially Penetrating
Coefficient of Permeability,

$$k = [2.303\ q\ Log_{10}(R / r_w)]\ /\ \pi C[(H-s)^2 - t^2]$$

C = 1, nearly 1.0

s = length of un-penetrating depth

t = depth from draw-down to bottom

r_w = Radius of the well

R = Radius of the draw-down cylinder

EFFECTIVE STRESS AND SEEPAGE PRESSURE

No flow condition,

42. Total vertical pressure, $p = H_0 \gamma_w + z \gamma_{sat}$

43. Pore water pressure, $u_w = H_0 \gamma_w + z \gamma_w$

44. Effective vertical pressure, $\sigma' = p - u_w = z(\gamma_{sat} - \gamma_w) = z\gamma'$

z = certain depth of the soil

Downward flow condition,

45. Pore water pressure, $u_w = z\,(H_0 + H_s - h)\gamma_w \,/\, H_s$

46. Total vertical pressure, $\sigma' = zh\gamma_w \,/ H_s$

H_s = total depth of the soil, h= depth down

47. Effective vertical pressure, $\sigma' = z\gamma' + iz\gamma_w$

Upward flow condition,

48. Pore water pressure, $u_w = z\,(H_0 + H_s + h)\gamma_w \,/\, H_s$

49. Effective vertical pressure, $\sigma' = z\gamma' - iz\gamma_w$

50. Critical Hydraulic gradient, $i_c = \gamma'/\gamma_w$ where, $\sigma'=0$

SEEPAGE OF WATER THROUGH SOILS

Flow net in isotropic soil,

51. Total quantity of water flow under dam, sheet pile,

$q_t = kH(N_f / N_d)$

N_f = number of flow channels in the net

N_d = number of equipotential drop

H= Head difference

Flow net in Anisotropic soil,

51. Total quantity of water flow under dam, sheet pile,

$q_t = \sqrt{(k_x \,.\, k_z)}\,h(N_f / N_d)$

52. Seepage line- free Surface, $a = (d/\cos_{\beta}) - \sqrt{(d^2/\cos^2_{\beta} - h^2/\sin^2_{\beta})}$

Heaving of soil at Exit Point

53. The pore water pressure at certain point A,

$u_A = \gamma_w \{z_A + d_w + (\text{rest of } N_d \text{ at point A} / N_d)h\}$

Like, $u_A = \gamma_w \{z_A + d_w + (2/9)h\}$ **(at tailwater side)**

$z_A =$ **Depth of soil Point A to top of the soil (at tailwater side)**

$d_w =$ **Depth of water from top of the soil to water level(at tailwater side)**

Factor of safety for sheet pile against heave or boiling of the soil

Where, i = Hydraulic gradient, h/L is too high.

54. Factor of safety, $FS = W'/U = \gamma'/(i_{av}.\gamma_w)$,

where, $\gamma' = (\gamma_{sat} - \gamma_w) \times h$,
h=depth of heave soil prism/unit length pile.
i_{av} = N_d at middle of heave soil prism /unit length pile.
W'= Submerged weight of soil in the heave zone per unit width of sheet pile
U= Uplift force due to seepage on the same volume of soil

$W' = D^2(\gamma_{sat} - \gamma_w)/2 = D^2\gamma'/2$,
Where, D= is the depth of embedment into Permeable soil
$U = D^2(i_{av}.\gamma_w)/2$ Block of heave soil = D/2 x D, max heave within D/2 from sheet pile

COMPRESSIBILITY OF SOIL AND ROCK

Vertical stress under Foundation

Vertical pressure on each layer,
55. $\Delta p = (\Delta p_t + 4\Delta p_m + \Delta p_b)/6$
Δp_t, Δp_m, Δp_b are the increase in pressure at top, middle, bottom

56. Avarage Vertical pressure,
$\Delta p_{av} = (\Delta p_A + 4\Delta p_B + \Delta p_C)/6$
Δp_A, Δp_A, Δp_C are the pressure at LAYER

Time rate Consolidation, Settlement

57. compression index, $C_c = 0.009(LL-10)$

58. swell index, $C_s = 1/5$ to $1/6$

59. Settlement, $S = H\,\Delta e/(1+e_0)$, For One-dimensional consolidation

60. Settlement, $S = C_c H[\log(p_0+\Delta p)/p_0]/(1+e_0)$,
For $p_0 = p_c$, normal consolidated clay
p_0 = Effective overberden pressure
p_c = Preconsolidation pressure

61. Pre-consolidation pressure,
$P_c = .5q_u/(.11+.0037\,PI)$

62. Settlement, $S = C_s H [\log(p_0+\Delta p_1) / p_0] / (1+e_0)$,
For $(p_0+\Delta p) <= p_c$, pre-consolidated clay

63. Settlement, $S = [C_s H \log(p_0 / p_c) + C_c H \log(p_0+\Delta p_1)] / (1+e_0)$
For $p_0 < p_c < (p_0+\Delta p)$
Δp=pressure increment, $e_0 =$ initial void ratio

64. Coefficient of Consolidation,
$T_v = c_v t / H_d^2$
T_v = Time factor, see table 10.3 book(CES) page-10-15
$H_d = H/2$ = Half thickness of soil layer for two way drainage

65. Coefficient of Consolidation,
$c_v = k / \gamma_w [(\Delta e+\Delta p) / (1+e_0)]$

STRENGTH OF SOIL AND ROCK

66. Normal stress on the shear plane,
$\sigma_\theta = (\sigma_1 + \sigma_3)/2 + \cos 2\theta (\sigma_1 - \sigma_3)/2$

67. Shearing Strength parallel to the plane,
$\tau_\theta = \sin 2\theta (\sigma_1 - \sigma_3)/2$

$\Theta = 45+\Phi/2$, Angle make with failure plane

68. Shearing Strength, $\tau = c + \sigma \tan\Phi$
Φ = Angle of internal friction
σ = Normal force
c = Cohesion of the soil

69. Friction angle Φ, $\sin\Phi = [(\sigma_1-\sigma_3)/2] / [(\sigma_1+\sigma_3)/2 + c/\tan\Phi]$

70. Saturated soil, $\tau = c + (\sigma-u) \tan\Phi = c + \sigma' \tan\Phi$

71. Partially Saturated soil, $\tau = c + (\sigma_n-u_a) \tan\Phi + (u_a-u_w) \tan\Phi_b$

72. Major Principal stress, $\sigma_1 = \sigma_3 . \tan^2(45+\Phi/2) + 2c . \tan(45+\Phi/2)$,
$\sigma_3 =$ Minor principal stress

72. Sensitivity of clay, $S_t =$ Undisturbed q_u / Remolded q_u

73. Pore-pressure coefficient, $B = \Delta u / \Delta \sigma_3$.
Δu, Pore-water Pressure Due to Change in All-around Stress.

B equals one for saturated soil and rock materials. For intact rocks, where *C* may approach *Cs*, *B* is less than one. Values of *B* on the order of 0.5 are found in partially saturated soils.

ENGINEERING GEOLOGY OF THE ROCKS AND SOIL

74. **Earthquake**, Lateral force, $V=ZIKCSW$

Where, Z = zone factor,
I=intensity=1, 1.5 for Hospital
K=0.67, Space Frame K=0.80, Frame / shear wall K=1,
Shear wall Box K=1.33
$C=1/(15\sqrt{T})$, $T=0.1N$, No. of floor
S=1 or 1.5 for Rock foundation
W= Total Building dead load plus 25% floor live load.

ENGINEERING SUBSURFACE INVESTIGATION

Field Vane Shear Test

75. Torque, $T=px=\prod C_u(d^2h/2)+(d^3/6)$

Cu= 1.7- 0.54 *(PI)* where *C* Correction factor, *PI* Plasticity index of the soil.

Standard Penetration test,

76. Corrected N-value, $N_1(60) = N \times C_e \times C_l \times C_s \times C_d \times C_N$
$C_N = \sqrt{(p / \sigma_v)}$
where *P* 100 kPa or 2.0 ksf or 1 tsf, or 1 kg/cm 2

where *(*N1)60 =Normalized SPT blow count, for 60% rod-energy ratio and 100
kPa (1 kg/cm 2 ; 1 tsf, 2 ksf)
N= Field SPT blow count, from 6 to 18 inches
C_e= Correction for hammer release system energy
C_l = Correction for rod length
C_s =Correction for sampler type
C_d =Correction for bore hole diameter
C_N =Correction for effective overburden pressure

Static-Cone Penetration Test

A rod, having an enlarged cone-shaped tip of 1.4 inches diameter, is pushed into theground at the rate of 2 to 4 feet per minute of the soils encountered. An empirical relationship between normalized cone resistance, normalized friction ratio, and soil identification is.

77. $q_{c1e} = q_c / (\sigma_v')_c$
78. $f_{c1e} = fs/ (\sigma_v')_s$
79. $R_f = 100\,(fs/q_c)$

where σ_v'= Vertical effective stress (1 atm, 1 tsf, or 100 kPa)
qc1e =Normalized cone resistance
qc =Measured cone resistance (1 atm, 1 tsf, or 100 kPa)

c= Cone resistance stress exponent
f_{s1e} =Normalized sleeve friction
f_s =Measured sleeve friction (1 atm, 1 tsf, or 100 kPa)
R_f =Friction ratio, percent.

Estimating Relative Density and Friction Angle from SPT Data

Presented empirical relationships that can be reasonably approximated by a straight line for N-values up to 50 blows per foot (0.3 m):

80. For coarse-grained sands: $\Phi' = 30° N/3$
81. For fine-grained sands: $\Phi' = 28° N/4$

Estimating Unconfined Compressive Strength from CPT Data

82. $Su = (qc - \sigma_{total}) / Nk$
where Su =Untrained cohesive strength
qc =Measured CPT cone resistance
σ_{total} / =In situ total overburden stress
Nk =Empirical untrained strength-bearing factor.
This equation is applicable for most sedimentary, non-sensitive clays.

Estimating Drained Friction Angle from CPT Data

There are two methods for estimating the drained friction angle of clean sands: An empirical correlation that indicates

83. $\Phi' = 28° + 12.4 \log(q_{c1e})$
Where, the normalized tip resistance, $qc1e$, measured in MPa,

Estimating Pre-consolidation Pressure

84. Effective overburden pressure,
$P'_c = .5q_u / (.11+.0037 PI)$; $C_u = .5q_u$

Estimation of Liquefaction Potential

85. $\tau_{cyc} = 0.65 a_{max} \sigma_v rd / g$

where $\tau_{cyc=}$ Uniform cyclic shear stress
a_{max} =Peak ground surface acceleration
g= Acceleration of gravity
σ_v =Total vertical stress
rd= Stress reduction factor (see Figure 7.25).

The Cyclic Stress Ratio is defined as

86. $CSR = \tau_{cyc} / \sigma_{v0}$, F.S= τ_{cyc}/Earth quake shear stress

SHALLOW FOUNDATION FOOTING AND RAFT

87. Ultimate Bearing Capacity,

$q_d = cN_c + \gamma D_f N_q + 0.5\gamma B N_\gamma$ For Continuous footing
C= Cohesion
γ= Unit wt. Of soil
D_f= Depth of foundation
B = Width of foundation
N_γ, N_c, N_q = Bearing capacity factor

88. Bearing Capacity,

$q_{dr} = 1.2cN_c + \gamma D_f N_q + 0.6R\gamma N_\gamma$ For Circular footing on hard soil

89. Bearing Capacity,

$q_{dr} = 1.2cN_c + \gamma D_f N_q + 0.4B\gamma N_\gamma$ For Square (BxB) footing on hard soil

90. Bearing Capacity,

$q_{ult} = cN_{cq} + .5B\gamma N_{\gamma q}$ For Continuous footing with inclined load

Continuous Footing at top of slope and on a slope (Case-I and Case-II)

91. Bearing Capacity, $q_{ult} = cN_{cq} + .5\gamma BN_{\gamma q}$
For Continuous footing with water level $d_o >= B$

92. Bearing Capacity, $q_{ult} = cN_{cq} + .5B\gamma_{sub}N_{\gamma q}$
For Continuous footing with water level at GL
Using 0.4B for squre and 0.6R for circular footing instate of 0.5B

Bearing Capacity of Cohesive Soils

Single Cohesive Layer.

93. The ultimate bearing capacity of cohesive soils, $q_d = cN_c + \gamma D_f$
$q_{d(net)} = cN_c$

For a continuous footing, for *Df / B* <= 4
$Nc = 5.14 + [(Df / B) / 0.37 + 0.35 (Df / B)]$

For a circular or square footing, for *Df / B* <= 4
$Nc = 6.2 + [(Df / B) / 0.45 + 0.24 (Df / B)]$

For a rectangular footing,
$Nc = (0.84 + 0.16\, B / L)\, Nc$ (square)

DEEP FOUNDATION PILES AND PIERS

Ultimate vertical load capacity of pile or pier

94. $Qult = Qb + Qs - Wp = 9c_uA_p + \alpha\, c_u\, p\, L$

Where, $Qult$ = Ultimate vertical load capacity of pile or pier
Qb = Component of load capacity due to bearing capacity at pile or pier base
Qs = Component of load capacity due to side friction
α = adhesion factor,
p=perimeter
L=Length

Other method

Load capacity at pile or pier base

95. $Q_b = A_b(cNc + \sigma_t' Nq \_ 0.5 B\gamma b' N_\gamma)$

Where A_b = Area of pile or pier base
c = Soil cohesion
σ_t'=Effective vertical stress at pile or pier base
B=Base diameter
$\gamma b'$=Effective unit weight of soil in the failure zone beneath base
Nc, Nq, N_γ = Bearing capacity factors. Page-8.4 Fig-8.4

The load capacity due to skin friction on the shaft of the pile

96. $Qs = \Sigma\, \sigma_t'\, K_{hc} \tan\delta PL$,
Where, σ_t' = Effective overburden pressure
K_{hc} = Ratio of horizontal to vertical pressure–pile in compression
δ = friction angle between pile and soil (see Table 9.4)
P= Perimeter or circumference of pile, For circular pile, $P=\pi D$
L= length of the pile.

Carrying Capacity of a Single Pile or Pier in Granular Soil

97. $Qult = Ab\, \sigma_t'\, Nq + \Sigma\, \sigma_t'\, K_{hc} \tan\delta PL$, Where, $c=0$, $N_\gamma = 0$

Carrying Capacity of a Single Pile or Pier in Cohesive Soil

98. $Qb\text{-}ult = A_b c\, N_c$ Where, $c = .5q_u$, $N_q = 0$ and $\delta = 0$

Skin Friction factor for Driven Piles.

99. $Qs\text{-}ult = \Sigma\, \alpha\, c_u\, PL$,
$\alpha = ca / cu = 1.0$, $cu \leq 0.25$ tsf
$\alpha = ca / cu = 1.25 - cu$, $0.25 < cu < 0.75$ tsf
$\alpha = ca / cu = 0.5$, $cu > 0.75$ tsf

Settlement of Pile Groups

Pile Group in Granular Soil.

100. $Sg = Si\sqrt{(B / D)}$

where Sg = Settlement of pile group
Si = Settlement of a single pile estimated or determined from load tests
B = Smallest dimension of pile group
D = Diameter of individual pile.

Displacement

101. $\delta_e = PL / AE$

Where, δ_e = Elastic compression
P = Axial load on pile
L = Pile length (for end-bearing pile)
A = Cross-sectional area of pile material
E = Modulus of elasticity of pile material.

102. $Sf = \delta_e + (0.15 + D/120)$,

where, Sf = Displacement at failure in inches,
δ_e = Elastic compression,
D = Pile diameter in inches

PILE CAPACITY FROM DRIVING DATA

Danish Formula

103. $Q_{dy} = aW_H H / (S + 0.5Se)$

$Se = \sqrt{(2aW_H HL / AE)}$

where Q_{dy} = Ultimate dynamic bearing capacity of driven pile
a=Pile driving hammer efficiency (normally 1)
W_H = Weight of hammer
H = Hammer drop (note that $W_H\ H$ = Hammer energy)
S = Inelastic set of pile, in distance per hammer blow
Se = Elastic set of pile, in distance per hammer blow
L = Pile length
A = Pile end area
E = Modulus of elasticity of pile material (see Equation 9.1).

RETAINING STRUCTURES

LATERAL EARTH PRESSURE

104. $\sigma'h = K_0\sigma'_v = K_0 (q_0 + \gamma H)$,

where $\sigma'h$ = Effective horizontal pressure
q_0 = *surcharge load*
σ'_v = Effective vertical pressure
$K_0 = 1 - \sin\Phi = \sigma'_v / \sigma'h$, coefficient of earth pressure at rest, generally 0.4 to 0.6.

$K_{0} = \sigma'_v / \sigma' h$,

Total Horizontal force,

105. $P_h = \gamma H^2 K_0 / 2$

106. $P_h = \gamma' H^2 K_0 / 2 + \gamma_w H^2 / 2$ **Where, C=0, Submerged condition**

107. $P_h = \gamma H^2 / 2 + 2cH$ **Where, Φ=0, tan45=1, untrained condition**

ACTIVE EARTH PRESSURE

108. $P_a = (\gamma H^2 K_a / 2) - 2cH\sqrt{k_a}$

Where,

$k_a = \cos^2(\alpha+\varphi) / \cos^2\alpha \cos(\alpha-\delta)[1+\sqrt{\{ \sin(\varphi+\delta) \sin(\varphi-\beta) / \cos(\alpha-\delta) \cos(\alpha+\beta)\}}]^2$

$k_a = \cos^2(\alpha+\varphi) / \cos^3\alpha\, [1+\sqrt{\{ \sin\varphi \sin(\varphi-\beta) / \cos\alpha \cos(\alpha+\beta)\}}]^2$

where, δ=0

For the simple case where the wall is vertical (α= 90°) and the backfill is horizontal(β= 0°):

109. $K_a = (1-\sin\varphi)/(1+\sin\varphi) = \tan^2(45-\varphi/2)$

COHESIVE SOILS (vertical Cut)

110. $H_o = 4c/\gamma = 2z_o$, where $\varphi = 0°$

Where z_o is the depth at which the pressure against a retaining wall is zero, where the active pressure diagram starts.

PASSIVE PRESSURE

111. $P_P = (\gamma H^2 K_p / 2) + 2c\sqrt{k_p}$

where,

$k_p = \cos^2(\alpha+\varphi) / \cos^2\alpha \cos(\delta-\alpha)[1+\sqrt{\{ \sin(\varphi-\delta) \sin(\varphi+\beta) / \cos(\delta-\alpha) \cos(\beta-\alpha)\}}]^2$

112. $K_p = (1+\sin\varphi)/(1-\sin\varphi) = \tan^2(45+\varphi/2)$

For the simple case where the wall is vertical (α= 90°) and the backfill is horizontal(β= 0°) and δ=0

113. **Retaining wall Factor of Safety,** $F.S. = (d\ 2t \tan\varphi) / (\sigma'_h / \sigma'_v)A$, $k = \sigma'_h / \sigma'_v$

A= Area of surface of the wall,
t=thickness of wall,
d=required cantilever or penetration depth of the wall