EWCE 343

Geotechnical and Foundation Engineering

Consolidation Soil Settlement

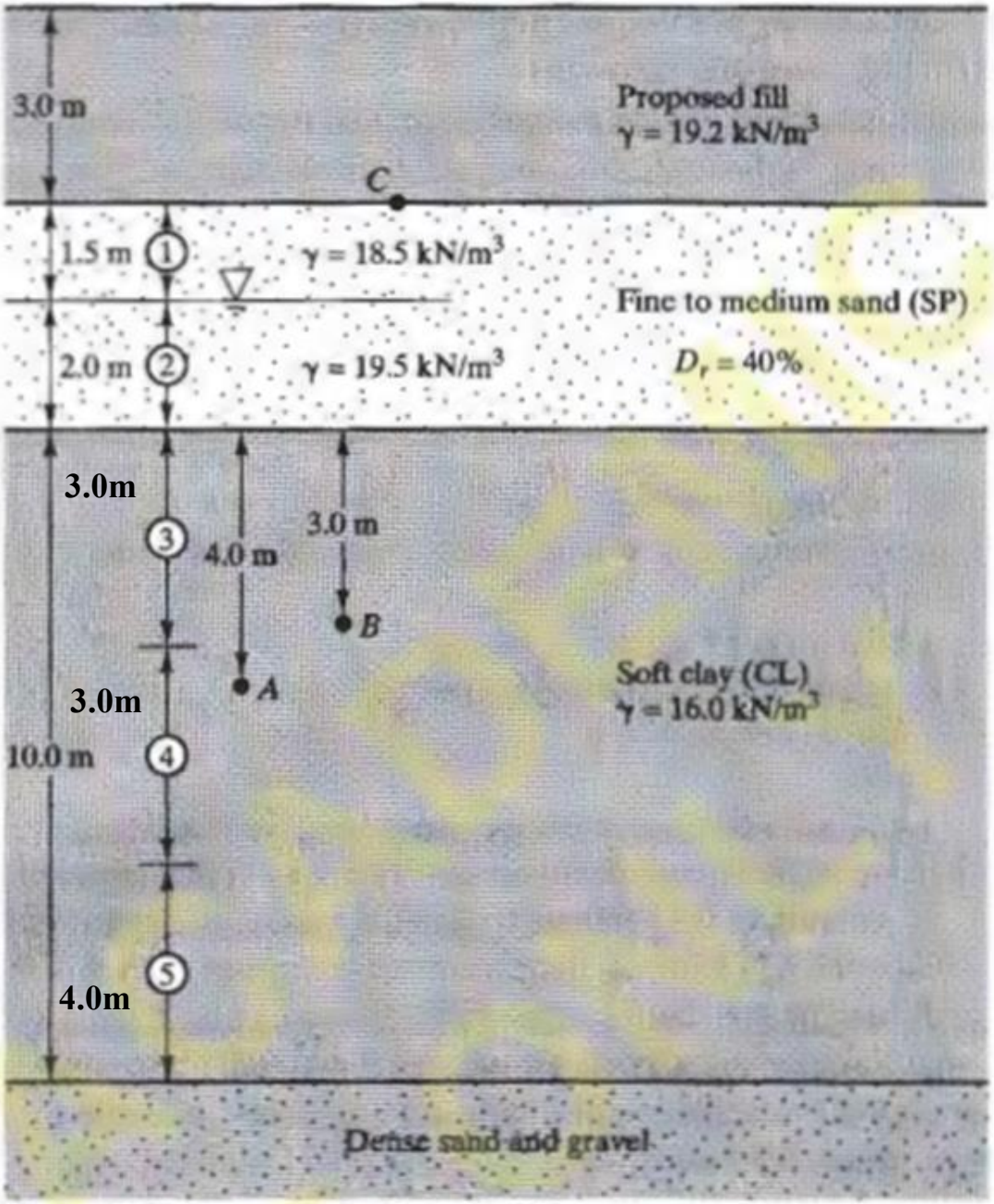


Figure 11.19 Soil profile for Example 11.5.

A consolidation test on a **sample** from **point A** produced the following results:

C_c=0.40

C_r=0.08

e_0=1.10

σ_c'=70 kpa

Sample is representative of the **entire soft clay stratum.**

Compute the **ultimate consolidation settlement due to the weight of the fill**.

Problem 1:

A 3.0 m deep compacted fill is to be placed over the soil profile shown in figure 1. A consolidation test on a sample from point A produced the following results:

C_c=0.40
C_r=0.08
e_0=1.10
σ_c'=70 kpa

Sample is representative of the entire soft clay stratum. Compute the ultimate consolidation settlement due to the weight of the fill.

Solution:

$$\sigma_f' = \sigma_{zo}' + \gamma_{fill} h_{fill}$$
$$= \sigma_{zo}' + (19.2)(3.0)$$
$$= \sigma_{zo}' + 57.6 \text{ kPa}$$

Compute the initial vertical stress at sample location:

$$\sigma_{zo}' = \sum \gamma h - u$$
$$= 18.5(1.5) + 19.5(2.0) + 16.0(4.0) - 9.81(6.0) = 72.0 \text{ kPa}$$
σ_c'=70 kPa

$$\frac{C_c}{1+e_0} = \frac{0.40}{1+1.10} = 0.190$$

At the sample $\sigma_c' \approx \sigma_{zo}'$ soil is normally consolidated

TABLE 11.3 TYPICAL CONSOLIDATION PROPERTIES OF SATURATED NORMALLY CONSOLIDATED SANDY SOILS AT VARIOUS RELATIVE DENSITIES (Adapted from Burmister, 1962)

Soil Type	$C_c/(1+e_o)$					
	$D_r = 0\%$	$D_r = 20\%$	$D_r = 40\%$	$D_r = 60\%$	$D_r = 80\%$	$D_r = 100\%$
Medium to coarse sand, some fine gravel (SW)	-	-	0.005	-	-	-
Medium to coarse sand (SW/SP)	0.010	0.008	0.006	0.005	0.003	0.002
Fine to coarse sand (SW)	0.011	0.009	0.007	0.005	0.003	0.002
Fine to medium sand (SW/SP)	0.013	0.010	0.008	0.006	0.004	0.003
Fine sand (SP)	0.015	0.013	0.010	0.008	0.005	0.003
Fine sand with trace fine to coarse silt (SP-SM)	-	-	0.011	-	-	-
Find sand with little fine to coarse silt (SM)	0.017	0.014	0.012	0.009	0.006	0.003
Fine sand with some fine to coarse silt (SM)	-	-	0.014	-	-	-

If the soil at **the sample depth is normally consolidated**, and the sample is truly representative, then the **entire stratum is normally consolidated (NC).**

Assume the sand also is NC, which is conservative.

For sand strata, Fine to medium sand (SP) and D_r= 40%

Table 11.3 (Coduto), $\frac{C_c}{1+e_0}$ = 0.008

$$\delta_{c.ult} = \sum \frac{C_c}{1+e_0} \text{H} \log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right) (1)$$

Layer	H (m)	At midpoint of layer			Equation	$\delta_{c.ult}$ (mm)
		σ'_{z0} (kPa)	σ'_{zf}(kPa)	$\frac{C_c}{1+e_0}$		
1	1.5	13.9	71.5	0.008	(1)	8
2	2.0	37.4	95.0	0.008	(1)	6
3	3.0	56.4	114.0	0.19	(1)	174
4	3.0	75.0	132.6	0.19	(1)	141
5	4.0	96.7	154.3	0.19	(1)	154
						$\delta_{c.ult}$= 483mm

Result, $\delta_{c.ult}$= 483mm

For sand strata, Fine to medium sand (SP) and D_r= 40%

Table 11.3 (Coduto), $\frac{C_c}{1+e_0}$ = 0.008

$$\delta_{c.ult} = \sum \frac{C_c}{1+e_0} \text{H} \log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right) \quad (1)$$

Layer	H (m)	At midpoint of layer			Equation	$\delta_{c.ult}$ (mm)
		σ'_{z0} (kPa)	σ'_{zf}(kPa)	$\frac{C_c}{1+e_0}$		

Result, $\delta_{c.ult}$= mm

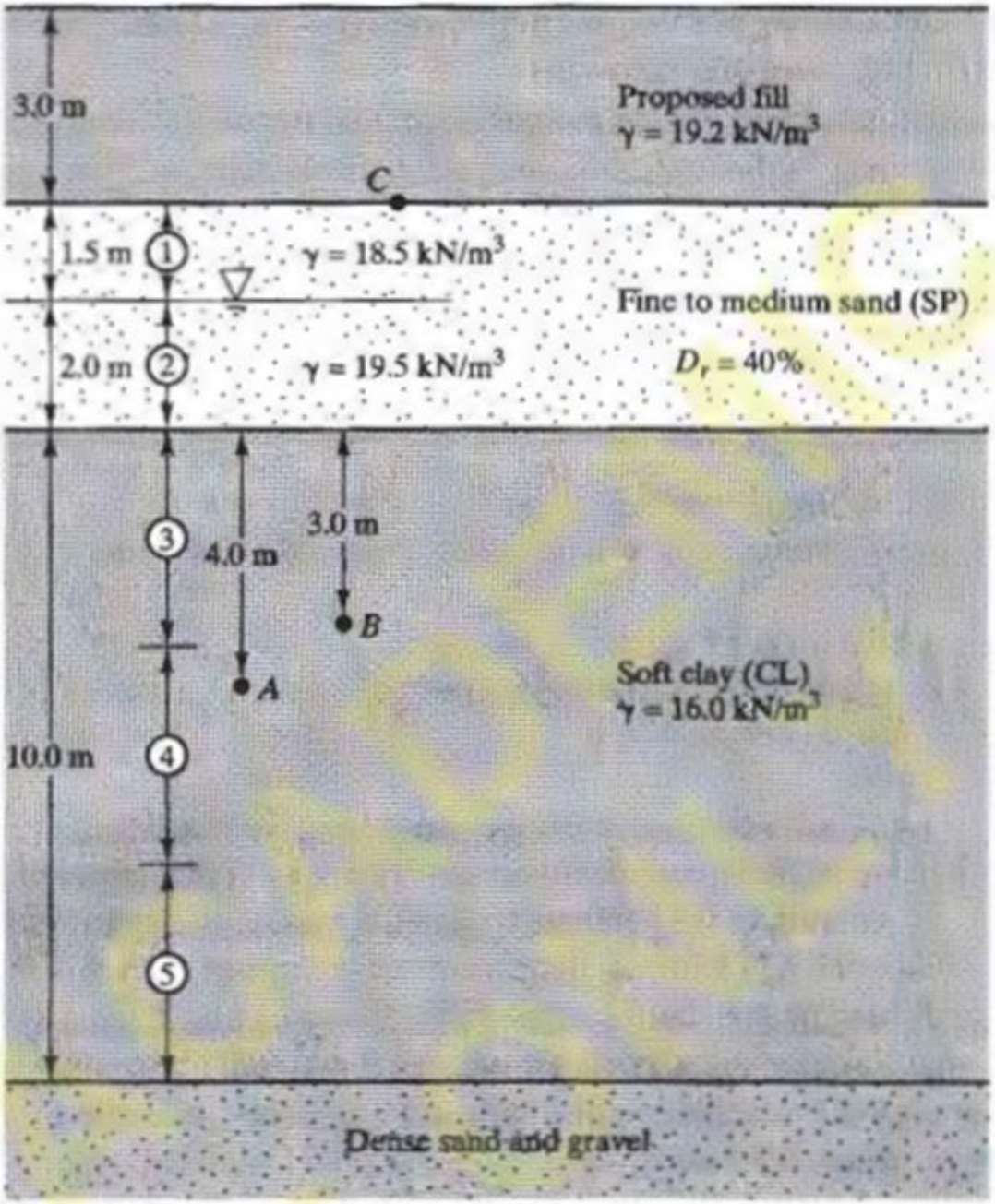


Figure 11.19 Soil profile for Example 11.5.

Problem 2: An 8.5m deep compacted fill is to be placed over the soil profile shown in the following figure. Consolidation tests on sample from points A and B produced the following results:

	Sample A	Sample B
C_c	0.25	0.20
C_r	0.08	0.06
e_0	0.66	0.45
σ_c'	101 kPa	510 kPa

Compute the ultimate consolidation settlement due to the weight of the fill.

Solution:

$$\sigma_{zf}' = \sigma_{zo}' + \gamma_{fill} h_{fill}$$
$$= \sigma_{zo}' + (20.3)(8.5)$$
$$= \sigma_{zo}' + 172.6 \text{ kPa}$$

Compute the initial vertical stress at sample location:

At sample A:

$$\sigma_{zo}' = \sum \gamma h - u$$
$$= 18.3(2.0) + 19.0(2.0) + 16.0(4.0) - 9.81(2.0) = 55.0 \text{ kPa}$$
$$\sigma_{zf}' = \sigma_{zo}' + 172.6 \text{ kPa}$$
$$= 55 + 172.6 = 227.6 \text{ kPa} \qquad \sigma_c' = 101 \text{ kPa}$$
$$\sigma_{zo}' < \sigma_c' \leq \sigma_{zf}' \qquad \text{Over-consolidated case II}$$

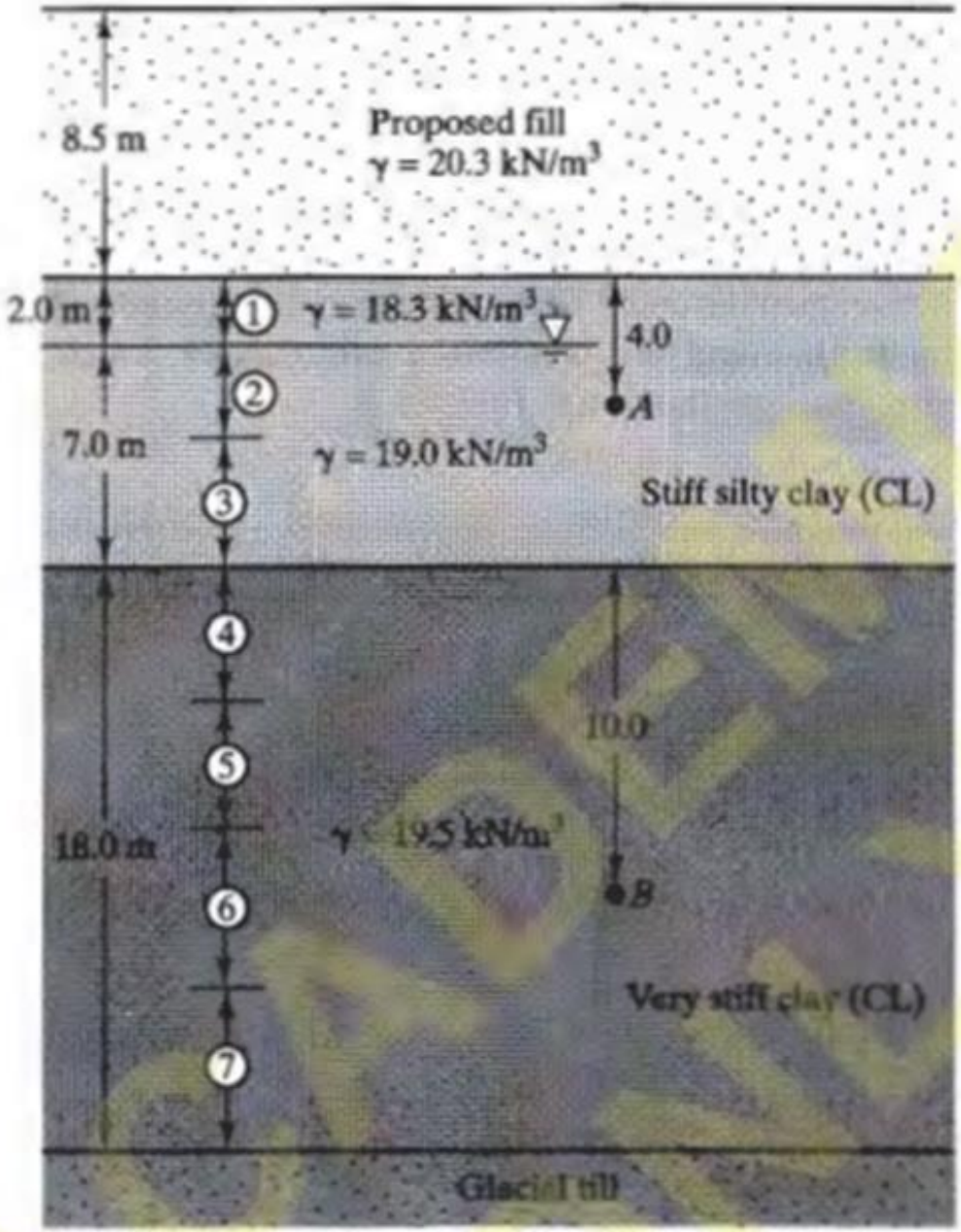


Figure 11.20 Soil profile for Example 11.6.

$$\delta_{c.ult} = \sum \frac{C_r}{1+e_0} \text{H} \log\left(\frac{\sigma'_c}{\sigma'_{z0}}\right) + \sum \frac{C_c}{1+e_0} \text{H} \log\left(\frac{\sigma'_{zf}}{\sigma'_c}\right) \quad \text{---------- (1)}$$

At sample B:

$\sigma'_{zo} = \sum \gamma h - u$

= 18.3(2.0) + 19.0(7.0)+ 19.5(10.0) – 9.81(17.0)= 198.0 kPa

$\sigma'_{zf} = \sigma'_{zo}$ + 172.6 kPa

=198.0+172.6 =370.6 kPa σ'_c=510 kPa

$\sigma'_{zo} < \sigma'_{zf} \leq \sigma'_c$ Over-consolidated case I

$$\delta_{c.ult} = \sum \frac{C_r}{1+e_0} \text{H} \log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right) \quad \text{---------- (2)}$$

Layer	H (m)	At midpoint layer			$\frac{C_r}{1+e_0}$	$\frac{C_c}{1+e_0}$	Equation	$\delta_{c.ult}$ (mm)
		σ'_{zo} kPa	σ'_c kPa	σ'_{zf} kPa				
1	2.0	18.3	64.3	190.9	0.05	0.15	1	196
2	3.0	50.4	96.4	223.0	0.05	0.15	1	206
3	4.0	82.6	128.6	255.2	0.05	0.15	1	217
4	4.0	120.4	-	293.0	0.04	0.14	2	62
5	4.0	159.2	-	331.8	0.04	0.14	2	51
6	5.0	202.8	-	375.4	0.04	0.14	2	53
7	5.0	251.4	-	424.0	0.04	0.14	2	45

$$\delta_{c.ult} = 830m$$

Note:

- Most of the compression occurs in upper stratum of Over Consolidated II (some of the compression occurs at virgin curve).
- Lower stratum, has much less compression even though it is twice as thick because it is over consolidated case I and ***all of the compression occurs on the recompression curve.***

Problem 3: The GWT in the soil profile shown in the following figure is currently at the elevation labeled 'initial'. A proposed dewatering project will cause it to drop to the elevation labeled as 'final'. Compute the resulting ultimate consolidation settlement.

A consolidation test performed on a sample from point a , produced the following results:

$\frac{C_c}{1+e_0} = 0.14$

$\frac{C_r}{1+e_0} = 0.06$

$\sigma_c' = 3000 \text{ lb}/ft^3$

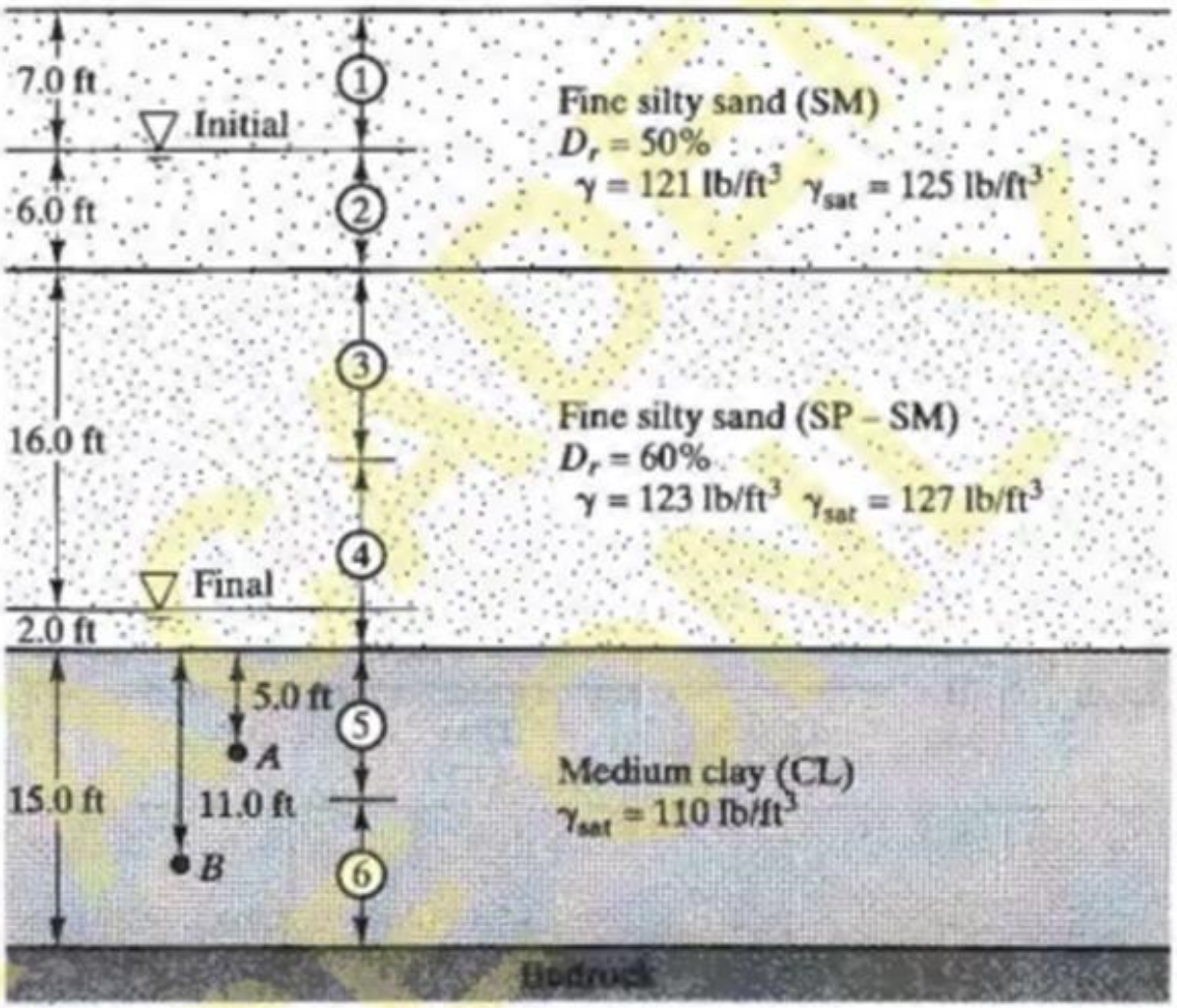


Figure 11.21 Soil profile for Example 11.7.

Solution:

At Sample A:

$$\sigma'_{z0}=\sum \gamma h - u$$

$$= 121x7 + 125x125 + 125x16 + 127 X 2 + 110x5 - 62.4x29 = 2623 \text{ lb}/ft^3$$

$\sigma'_{z0} \approx \sigma'_c$ Soil is normally consolidated.

Layer	H (ft)	At mid point of layer		$\frac{C_c}{1+e_0}$	Equation	$\delta_{c.ult}$ (inch)
		σ'_{z0} (lb/ft^3)	σ'_{zf} (lb/ft^3)			
1	7.0	423	423	0.011	NC	0.0
2	6.0	1035	1210	0.011	NC	0.1
3	9.0	1513	2126	0.09	NC	0.1
4	9.0	2095	3233	0.09	NC	0.2
5	7.0	2552	3837	0.14	NC	2.1
6	8.0	2909	4194	0.14	NC	2.1
					$\delta_{c.ult}$=	4.6

Problem 4: After the settlement due to the fill shown in the following figure is completed, a 20m diameter, 10m tall cylindrical steel water tank is to be built. The bottom of the tank will be at the top of the fill, and it will have an empty mass of 300.000 kg. Ultimately, the water inside will be 9.5m deep. Compute the ultimate consolidation settlement beneath the center of the tank due to the weight of the tank and its contents. Assume the new fill is over consolidated with $\frac{C_r}{1+ e_0}$.

Note:

- Settlement is now complete.
- σ'_{zf} is now σ'_{z0}

Solution:

W_{tank}= Mg = 300,000x9.8 ($\frac{1\ kN}{1000N}$) =2900 kN

W_{water}=$V_{tank}\gamma_w$= $\frac{1}{4}\pi B^2$H γ_w= $\frac{1}{4}$ (3.14x20^2x9.5 (9.81)= 29,200 kN

W=W_{tank}+W_{water}= 19200 +2900= 32,100 kN

A= $\frac{1}{4}\pi B^2$= $\frac{1}{4}\pi 20^2$=314 m^2

q = $\frac{W}{A}$ = $\frac{32100}{314}$ = 102 kPa

TABLE 2-7

Values or value ranges for Poisson's ratio μ

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand, gravelly sand	– 0.1–1.00
commonly used	0.3–0.4
Rock	0.1–0.4 (depends somewhat on type of rock)
Loess	0.1–0.3
Ice	0.36
Concrete	0.15
Steel	0.33

Figure 5-7 Influence factor I_F for footing at a depth D. Use actual footing width and depth dimension for thi
D/B ratio. Use program FFACTOR for values to avoid interpolation.

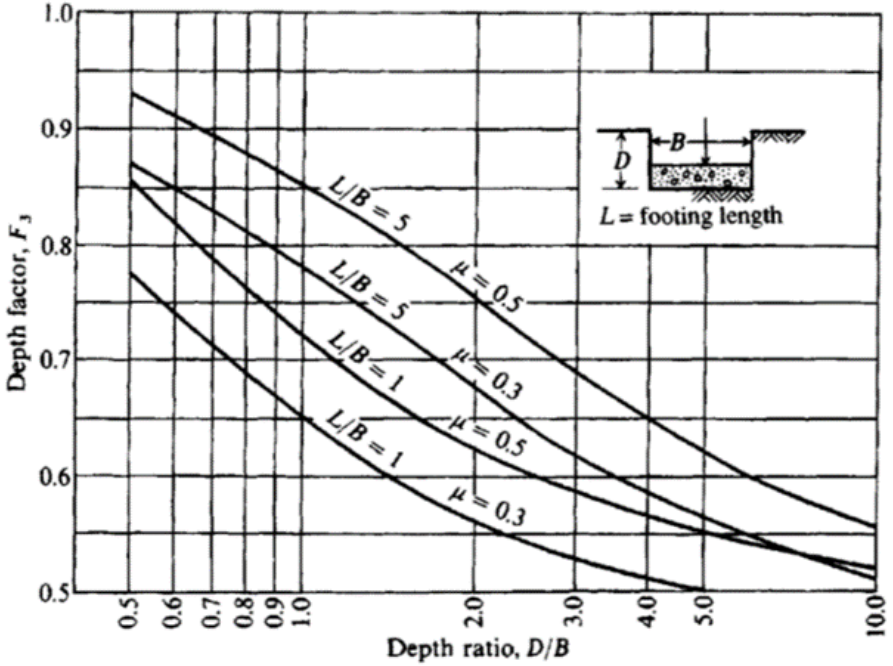


TABLE 5-2

Values of I_1 and I_2 to compute the Steinbrenner influence factor I_s for use in Eq. (5-16*a*) for several $N = H/B'$ and $M = L/B$ ratios

N		$M = 1.0$	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
0.2	I_1 =	0.009	0.008	0.008	0.008	0.008	0.008	0.007	0.007	0.007	0.007	0.007
	I_2 =	0.041	0.042	0.042	0.042	0.042	0.042	0.043	0.043	0.043	0.043	0.043
0.4		0.033	0.032	0.031	0.030	0.029	0.028	0.028	0.027	0.027	0.027	0.027
		0.066	0.068	0.069	0.070	0.070	0.071	0.071	0.072	0.072	0.073	0.073
0.6		0.066	0.064	0.063	0.061	0.060	0.059	0.058	0.057	0.056	0.056	0.055
		0.079	0.081	0.083	0.085	0.087	0.088	0.089	0.090	0.091	0.091	0.092
0.8		0.104	0.102	0.100	0.098	0.096	0.095	0.093	0.092	0.091	0.090	0.089
		0.083	0.087	0.090	0.093	0.095	0.097	0.098	0.100	0.101	0.102	0.103
1.0		0.142	0.140	0.138	0.136	0.134	0.132	0.130	0.129	0.127	0.126	0.125
		0.083	0.088	0.091	0.095	0.098	0.100	0.102	0.104	0.106	0.108	0.109
1.5		0.224	0.224	0.224	0.223	0.222	0.220	0.219	0.217	0.216	0.214	0.213
		0.075	0.080	0.084	0.089	0.093	0.096	0.099	0.102	0.105	0.108	0.110
2.0		0.285	0.288	0.290	0.292	0.292	0.292	0.292	0.292	0.291	0.290	0.289
		0.064	0.069	0.074	0.078	0.083	0.086	0.090	0.094	0.097	0.100	0.102
3.0		0.363	0.372	0.379	0.384	0.389	0.393	0.396	0.398	0.400	0.401	0.402
		0.048	0.052	0.056	0.060	0.064	0.068	0.071	0.075	0.078	0.081	0.084
4.0		0.408	0.421	0.431	0.440	0.448	0.455	0.460	0.465	0.469	0.473	0.476
		0.037	0.041	0.044	0.048	0.051	0.054	0.057	0.060	0.063	0.066	0.069
5.0		0.437	0.452	0.465	0.477	0.487	0.496	0.503	0.510	0.516	0.522	0.526
		0.031	0.034	0.036	0.039	0.042	0.045	0.048	0.050	0.053	0.055	0.058
6.0		0.457	0.474	0.489	0.502	0.514	0.524	0.534	0.542	0.550	0.557	0.563
		0.026	0.028	0.031	0.033	0.036	0.038	0.040	0.043	0.045	0.047	0.050
7.0		0.471	0.490	0.506	0.520	0.533	0.545	0.556	0.566	0.575	0.583	0.590
		0.022	0.024	0.027	0.029	0.031	0.033	0.035	0.037	0.039	0.041	0.043
8.0		0.482	0.502	0.519	0.534	0.549	0.561	0.573	0.584	0.594	0.602	0.611
		0.020	0.022	0.023	0.025	0.027	0.029	0.031	0.033	0.035	0.036	0.038
9.0		0.491	0.511	0.529	0.545	0.560	0.574	0.587	0.598	0.609	0.618	0.627
		0.017	0.019	0.021	0.023	0.024	0.026	0.028	0.029	0.031	0.033	0.034
10.0		0.498	0.519	0.537	0.554	0.570	0.584	0.597	0.610	0.621	0.631	0.641
		0.016	0.017	0.019	0.020	0.022	0.023	0.025	0.027	0.028	0.030	0.031
20.0		0.529	0.553	0.575	0.595	0.614	0.631	0.647	0.662	0.677	0.690	0.702
		0.008	0.009	0.010	0.010	0.011	0.012	0.013	0.013	0.014	0.015	0.016
500.0		**0.560**	0.587	0.612	0.635	0.656	0.677	0.696	0.714	0.731	0.748	0.763
		0.000	0.000	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.001	0.001

TABLE 5-2

Values of I_1 and I_2 to compute the Steinbrenner influence factor I_s for use in Eq. (5-16*a*) for several $N = H/B'$ and $M = L/B$ ratios *(continued)*

N		$M = 2.5$	4.0	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
0.2	I_1 =	0.007	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
	I_2 =	0.043	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044
0.4		0.026	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024
		0.074	0.075	0.075	0.075	0.076	0.076	0.076	0.076	0.076	0.076	0.076
0.6		0.053	0.051	0.050	0.050	0.050	0.049	0.049	0.049	0.049	0.049	0.049
		0.094	0.097	0.097	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098
0.8		0.086	0.082	0.081	0.080	0.080	0.080	0.079	0.079	0.079	0.079	0.079
		0.107	0.111	0.112	0.113	0.113	0.113	0.113	0.114	0.114	0.114	0.114
1.0		0.121	0.115	0.113	0.112	0.112	0.112	0.111	0.111	0.110	0.110	0.110
		0.114	0.120	0.122	0.123	0.123	0.124	0.124	0.124	0.125	0.125	0.125
1.5		0.207	0.197	0.194	0.192	0.191	0.190	0.190	0.189	0.188	0.188	0.188
		0.118	0.130	0.134	0.136	0.137	0.138	0.138	0.139	0.140	0.140	0.140
2.0		0.284	0.271	0.267	0.264	0.262	0.261	0.260	0.259	0.257	0.256	0.256
		0.114	0.131	0.136	0.139	0.141	0.143	0.144	0.145	0.147	0.147	0.148
3.0		0.402	0.392	0.386	0.382	0.378	0.376	0.374	0.373	0.368	0.367	0.367
		0.097	0.122	0.131	0.137	0.141	0.144	0.145	0.147	0.152	0.153	0.154
4.0		0.484	0.484	0.479	0.474	0.470	0.466	0.464	0.462	0.453	0.451	0.451
		0.082	0.110	0.121	0.129	0.135	0.139	0.142	0.145	0.154	0.155	0.156
5.0		0.553	0.554	0.552	0.548	0.543	0.540	0.536	0.534	0.522	0.519	0.519
		0.070	0.098	0.111	0.120	0.128	0.133	0.137	0.140	0.154	0.156	0.157
6.0		0.585	0.609	0.610	0.608	0.604	0.601	0.598	0.595	0.579	0.576	0.575
		0.060	0.087	0.101	0.111	0.120	0.126	0.131	0.135	0.153	0.157	0.157
7.0		0.618	0.653	0.658	0.658	0.656	0.653	0.650	0.647	0.628	0.624	0.623
		0.053	0.078	0.092	0.103	0.112	0.119	0.125	0.129	0.152	0.157	0.158
8.0		0.643	0.688	0.697	0.700	0.700	0.698	0.695	0.692	0.672	0.666	0.665
		0.047	0.071	0.084	0.095	0.104	0.112	0.118	0.124	0.151	0.156	0.158
9.0		0.663	0.716	0.730	0.736	0.737	0.736	0.735	0.732	0.710	0.704	0.702
		0.042	0.064	0.077	0.088	0.097	0.105	0.112	0.118	0.149	0.156	0.158
10.0		0.679	0.740	0.758	0.766	0.770	0.770	0.770	0.768	0.745	0.738	0.735
		0.038	0.059	0.071	0.082	0.091	0.099	0.106	0.112	0.147	0.156	0.158
20.0		0.756	0.856	0.896	0.925	0.945	0.959	0.969	0.977	0.982	0.965	0.957
		0.020	0.031	0.039	0.046	0.053	0.059	0.065	0.071	0.124	0.148	0.156
500.0		0.832	0.977	1.046	1.102	1.150	1.191	1.227	1.259	1.532	1.721	1.879
		0.001	0.001	0.002	0.002	0.002	0.003	0.003	0.003	0.008	0.016	0.031

TABLE 5-6

Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$ — — — ‡$E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ *$E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	¶$E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	†$E_s = 40\,000 + 1050N$ $E_{s(\text{OCR})} \approx E_{s,\text{nc}}\sqrt{\text{OCR}}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use $\quad$ $^{\$}E_s' = 2.5q_c$ $2500 < q_c < 5000$ use $\quad E_s' = 4q_c + 5000$ where E_s' = constrained modulus $= \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

TABLE 2-8

Value range* for the static stress-strain modulus E_s for selected soils (see also Table 5-6)

Field values depend on stress history, water content, density, and age of deposit

Soil	E_s, MPa
Clay	
Very soft	2–15
Soft	5–25
Medium	15–50
Hard	50–100
Sandy	25–250
Glacial till	
Loose	10–150
Dense	150–720
Very dense	500–1440
Loess	15–60
Sand	
Silty	5–20
Loose	10–25
Dense	50–81
Sand and gravel	
Loose	50–150
Dense	100–200
Shale	150–5000
Silt	2–20

*Value range is too large to use an "average" value for design.

Soils Settlement:

Soils Settlement:

Soils Settlement:

- SPT is widely used to obtain the bearing capacity of soils directly.

- One of the earliest published relationships was that of **Terzaghi and Peck** (1967).

- **Meyerhof** (1956, 1974) published equations for computing the allowable bearing capacity for a 25-mm settlement.

- Considering the accumulation of field observations and the stated opinions of the authors and others, this **author adjusted the Meyerhof equations** for an approximate 50 percent increase in allowable bearing capacity to obtain the following:

$$q_a = \frac{N}{F_1} k_d \qquad B \leq F_4 \qquad (1)$$

$$q_a = \frac{N}{F_2} \left(\frac{B+F_3}{B}\right)^2 k_d \qquad B > F_4 \qquad (2)$$

Where, q_a = allowable bearing pressure for ΔH_0= 25-mm or 1-in. settlement, kPa.

$$\boldsymbol{k_d}=\mathbf{1}+\mathbf{0.33}\frac{\boldsymbol{D}}{\boldsymbol{B}} \leq \mathbf{1.33} \text{ (Meyerhof)}$$

F factors as follows(SI Unit):

	N_{55}	N'_{70}
F_1	0.05	0.04
F_2	0.08	0.06
F_3	0.3	0.06
F_4	1.2	0.06

In these equations the allowable soil pressure is for an assumed 25-mm settlement. In general the allowable pressure for any settlement ΔH_j is:

$$\boldsymbol{q'_a}=\frac{\boldsymbol{\Delta H_j}}{\boldsymbol{\Delta H_0}}$$

Where, $\boldsymbol{\Delta H_0}$=25mm

$\boldsymbol{\Delta H_j}$=actual settlement that can be tolerated, in millimeters

Example: Find q_a from the diagram from the following data:

N_{70}= 24, Footing depth, D=1.0m footing width, B=3.0m

Solution:

Here, B=3.0 and F_4=1.2

since, B> F_4 use $q_a = \frac{N_{55}}{F_2}\left(\frac{B+F_3}{B}\right)^2 k_d$

F_2=0.08, F_3=0.3

$N_{55} = \frac{70}{55} N_{70} = \frac{70}{55} x24 = 30 > 24$

$\boldsymbol{k_d = 1 + 0.33\frac{D}{B} \leq 1.33}$

$\boldsymbol{k_d = 1 + 0.33\frac{1}{3} = 1.11 < 1.33}$

Substituting value in Following equation:

$$q_a = \frac{N_{55}}{F_2}\left(\frac{B+F_3}{B}\right)^2 k_d$$

$$= \frac{30}{0.08}\left(\frac{3.0+0.3}{3.0}\right)^2 1.11$$

$$= 503.66 \approx 500 \text{ kPa}$$

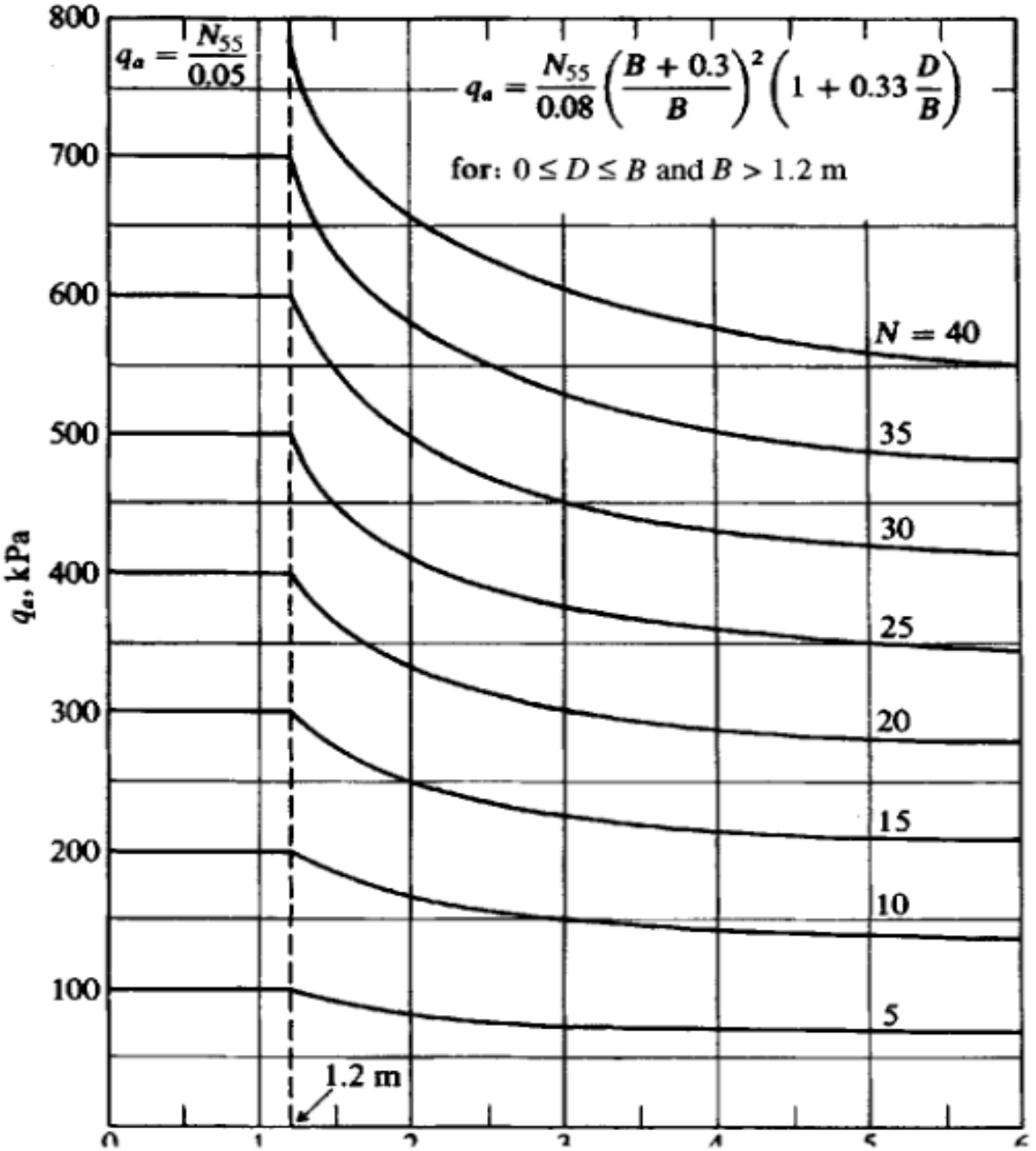


Figure 4-7 Allowable bearing capacity for **surface-loaded** footings with settlement limited to approximately 25 mm. Equation used is shown on figure.

Problem 4-8.

The average N'_{70}Nj0 blow count = 6 in the effective zone for a footing located at D = 1.6 m (blow count average in range from 1- to 4-m depth).

Required. What is the allowable bearing capacity for a 40-mm settlement?

Assume B> $\boldsymbol{F_4}$ where F_4=1.2

Use equation $\boldsymbol{q_a = \frac{N_{55}}{F_2}\left(\frac{B+F_3}{B}\right)^2 k_d}$

with **F2 = 0.08** and F3 = **0.30**

$$\boldsymbol{N_{55} = \frac{70}{55} N_{70} = \frac{70}{55} x6 = 9 > 6}$$

$$\boldsymbol{k_d = 1 + 0.33\frac{D}{B} \leq 1.33}$$

For 25mm settlement

Let B=1.5

$$q_a = \frac{N_{55}}{F_2}\left(\frac{B+F_3}{B}\right)^2 k_d \qquad k_d = 1 + 0.33\frac{1.6}{1.5} = 1.352 > 1.33$$

use k_d=1.33

$$q_a = \frac{9}{0.08}\left(\frac{1.5+0.3}{1.5}\right)^2 x\,1.33 = 215.46 \text{ kpa}$$

For **B=3.0**

$$q_a = \frac{N_{55}}{F_2}\left(\frac{B+F_3}{B}\right)^2 k_d \qquad k_d = 1 + 0.33\frac{1.6}{3.0} = 1.176 < 1.33 \text{ 0k}$$

$$q_a = \frac{N_{55}}{F_2}\left(\frac{B+F_3}{B}\right)^2 k_d$$

$$= \frac{9}{0.08}\left(\frac{3.0+0.3}{3}\right)^2 1.176 = \mathbf{160.083 \text{ kpa}}$$

Suggested $\boldsymbol{q_a}$=200 kPa for 25mm settlement

For 40mm settlement

$$\boldsymbol{q'_a}=\frac{\boldsymbol{\Delta H_j}}{\boldsymbol{\Delta H_0}} \text{ x } \boldsymbol{q_a}= \frac{\boldsymbol{40}}{\boldsymbol{25}}\text{x200=320 kpa}$$

Thanks