Introduction to Settlement and Volume Expansion in Soils

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An Introduction to Settlement and Volume Expansion in Soils

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1. INTRODUCTION

1.1 SCOPE. This publication concerns (a) immediate settlements, (b) long-term settlements, (c) rate of settlement, (d) criteria for tolerable settlement, (e) methods of reducing or accelerating settlements for saturated fine-grained soils and (f) methods for controlling and/or estimating heave in swelling soils. Procedures given are for fine-grained compressible soils as well as for coarse-grained soils.

1.2 OCCURRENCE OF SETTLEMENTS. The settlement of saturated cohesive soil consists of the sum of three components; (1) immediate settlement occurring as the load is applied, (2) consolidation settlement occurring gradually as excess pore pressures generated by loads are dissipated, and (3) secondary compression essentially controlled by the composition and structure of the soil skeleton. The settlement of coarse-grained granular soils subjected to foundation loads occurs primarily from the compression of the soil skeleton due to rearrangement of particles. The permeability of coarse-grained soil is large enough to justify the assumption of immediate excess pore pressure dissipation upon application of load. Settlement of coarse-grained soil can also be induced by vibratory ground motion due to earthquakes, blasting or machinery, or by soaking and submergence.

1.3 APPLICABILITY. Settlement estimates discussed in this publication are applicable to cases where shear stresses are well below the shear strength of the soil.

2. ANALYSIS OF STRESS CONDITIONS

2.1 MECHANICS OF CONSOLIDATION. See Figure 1. Superimposed loads develop pore pressures in compressible strata exceeding the original hydrostatic pressures. As pore pressure gradients force water from a compressible stratum, its volume decreases, causing settlement.

2.2 INITIAL STRESSES. See Figure 2 for profiles of vertical stress in a compressible stratum prior to construction. For equilibrium conditions with no excess hydrostatic pressures, compute vertical effective stress as shown in Case 1, Figure 2.

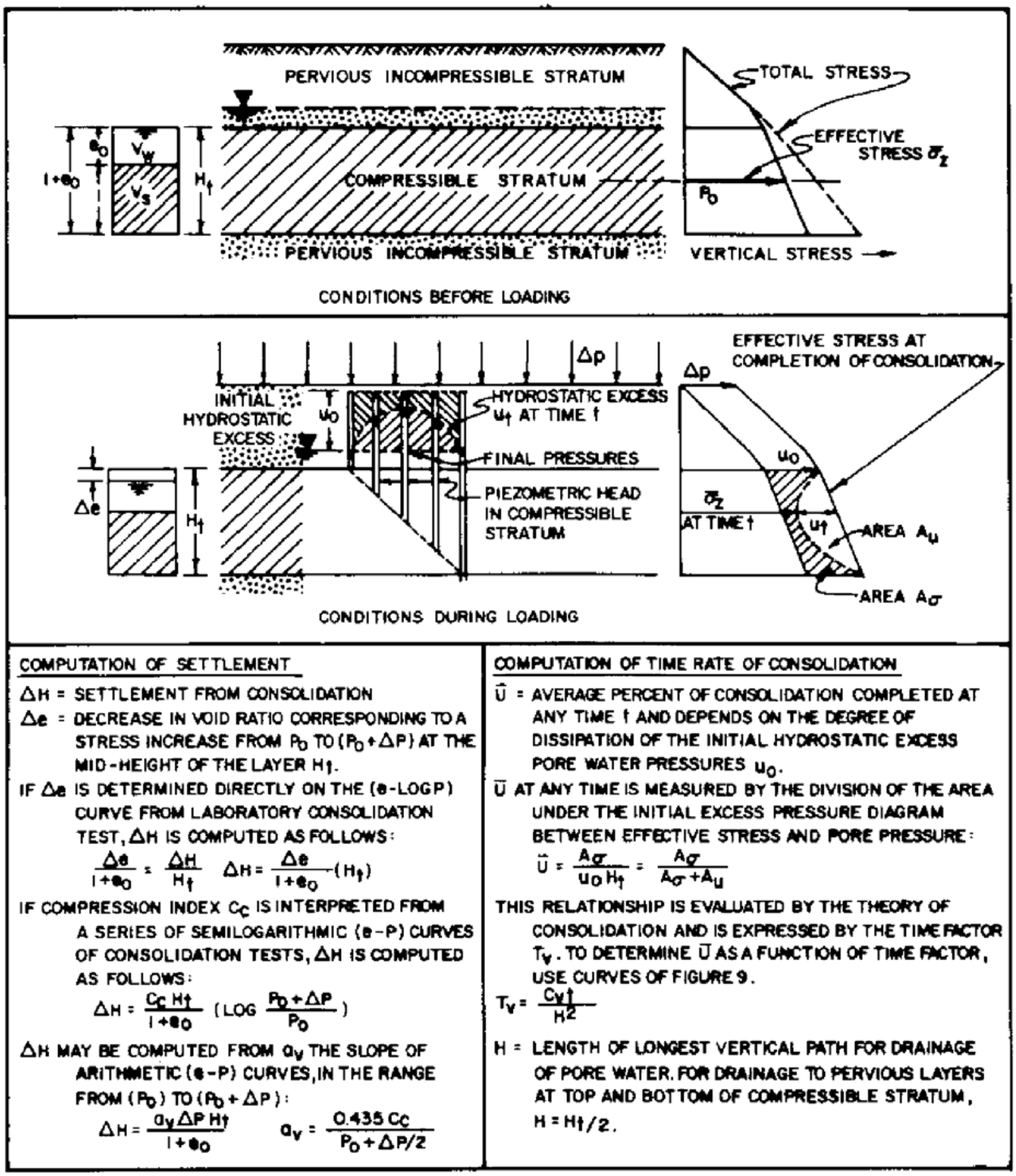


FIGURE 1
Consolidation Settlement Analysis

Figure 1

Consolidation Settlement Analysis

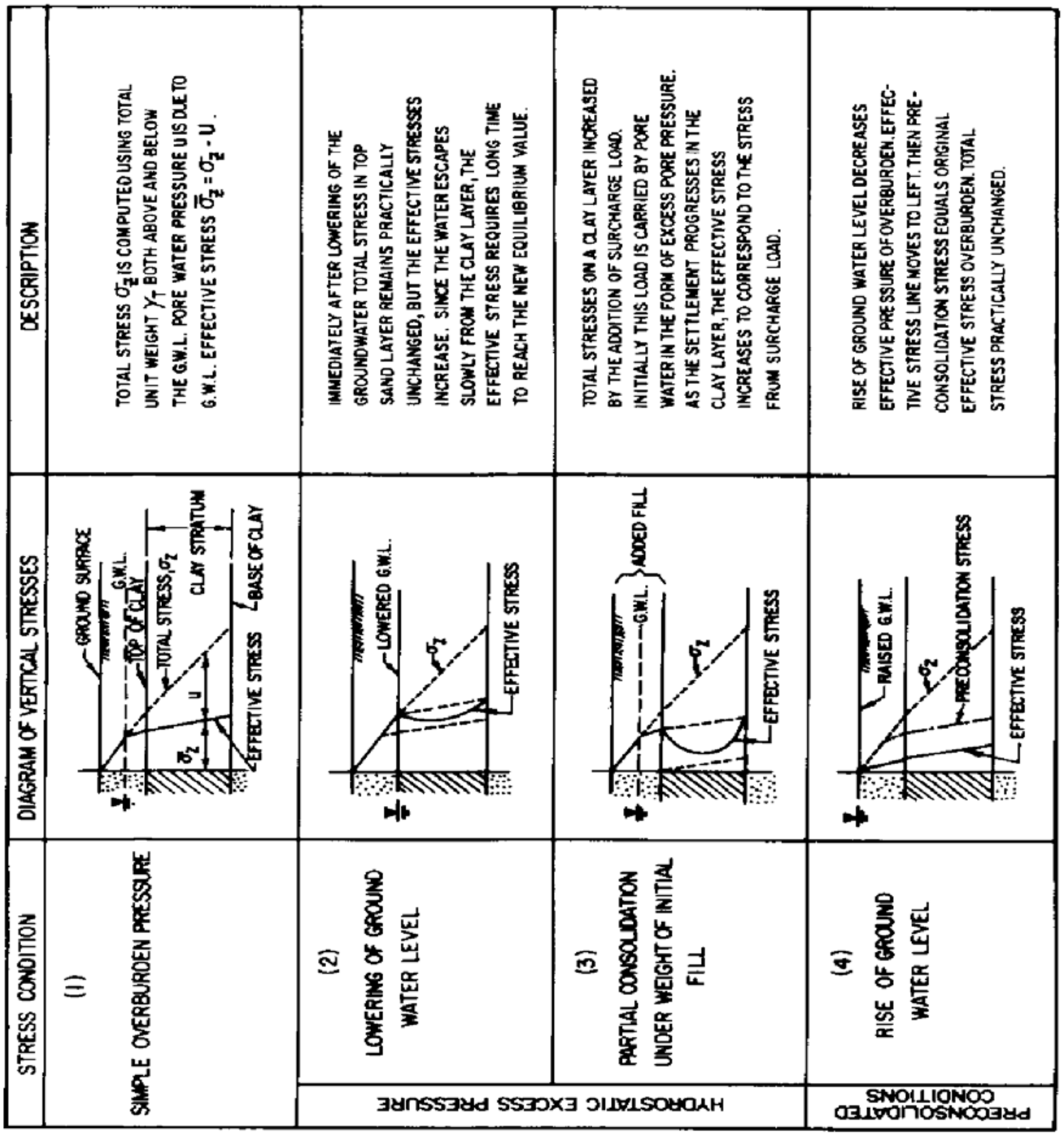
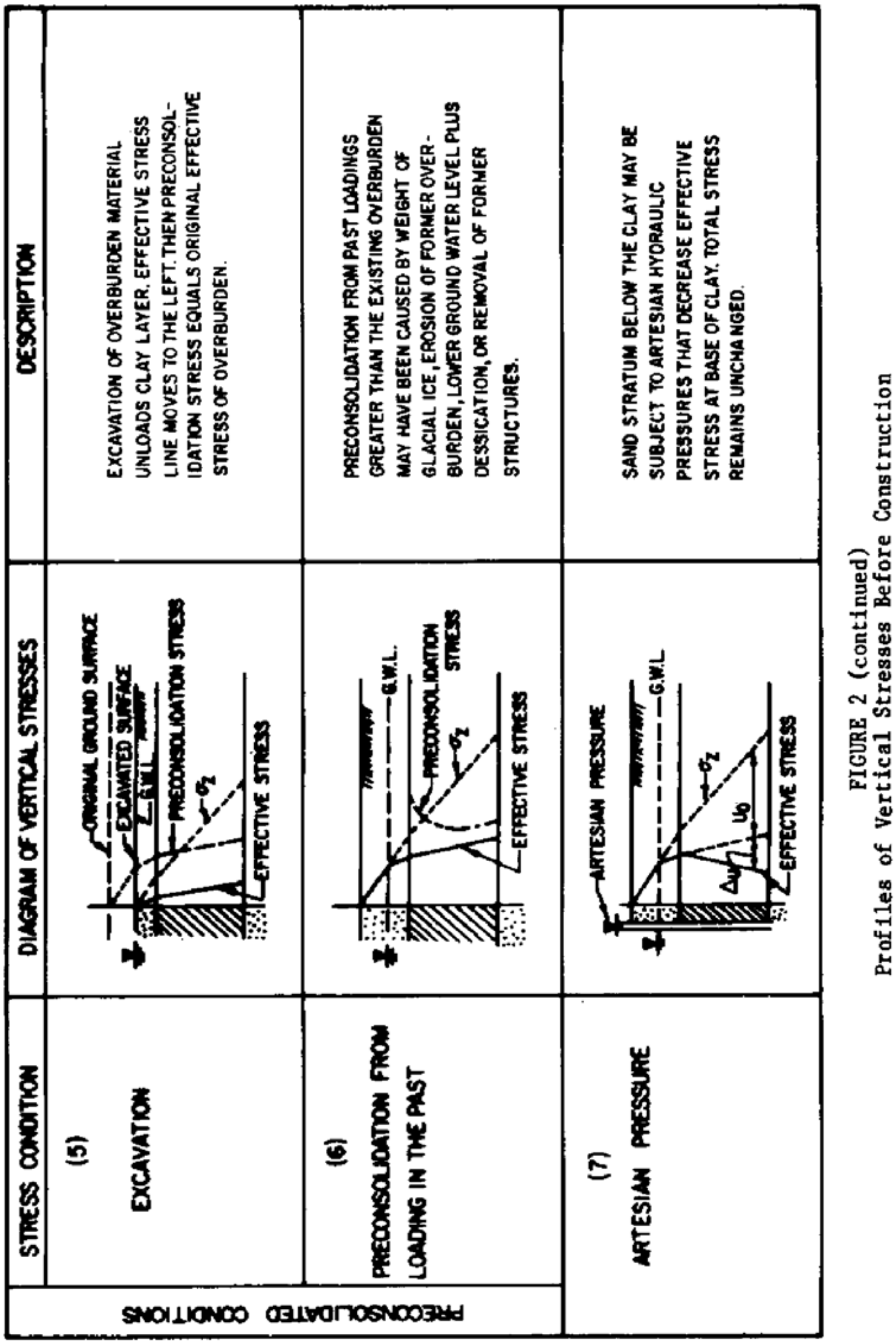


FIGURE 2
Profiles of Vertical Stresses Before Construction

Figure 2

Profiles of Vertical Stresses Before Compaction



	STRESS CONDITION	DIAGRAM OF VERTICAL STRESSES	DESCRIPTION
PRECONSOLIDATED CONDITIONS	(5) EXCAVATION	ORIGINAL GROUND SURFACE; EXCAVATED SURFACE; G.W.L.; PRECONSOLIDATION STRESS; σ_z; EFFECTIVE STRESS	EXCAVATION OF OVERBURDEN MATERIAL UNLOADS CLAY LAYER. EFFECTIVE STRESS LINE MOVES TO THE LEFT. THEN PRECONSOLIDATION STRESS EQUALS ORIGINAL EFFECTIVE STRESS OF OVERBURDEN.
	(6) PRECONSOLIDATION FROM LOADING IN THE PAST	G.W.L.; PRECONSOLIDATION STRESS; σ_z; EFFECTIVE STRESS	PRECONSOLIDATION FROM PAST LOADINGS GREATER THAN THE EXISTING OVERBURDEN MAY HAVE BEEN CAUSED BY WEIGHT OF GLACIAL ICE, EROSION OF FORMER OVERBURDEN, LOWER GROUND WATER LEVEL PLUS DESSICATION, OR REMOVAL OF FORMER STRUCTURES.
(7) ARTESIAN PRESSURE		ARTESIAN PRESSURE; G.W.L.; σ_z; u_0; Δu; EFFECTIVE STRESS	SAND STRATUM BELOW THE CLAY MAY BE SUBJECT TO ARTESIAN HYDRAULIC PRESSURES THAT DECREASE EFFECTIVE STRESS AT BASE OF CLAY. TOTAL STRESS REMAINS UNCHANGED.

FIGURE 2 (continued)
Profiles of Vertical Stresses Before Construction

Figure 2 (continued)

Profiles of Vertical Stresses Before Compaction

2.2.1 PRECONSOLIDATION. Stresses exceeding the present effective vertical pressure of overburden produce preconsolidation (1) by the weight of material that existed above the present ground surface and that has been removed by erosion, excavation, or recession of glaciers, (2) by capillary stresses from desiccation, and (3) by lower groundwater levels at some time in the past.

2.2.2 UNDERCONSOLIDATION. Compressible strata may be incompletely consolidated under existing loads as a result of recent lowering of groundwater or recent addition of fills or structural loads. Residual hydrostatic excess pore pressure existing in the compressible stratum will dissipate with time, causing settlements.

2.2.3 EVALUATION OF EXISTING CONDITIONS. Determine consolidation condition at start of construction by the following steps:

2.2.3.1 REVIEW THE DATA AVAILABLE on site history and geology to estimate probable preconsolidation or underconsolidation.

2.2.3.2 COMPARE PROFILE of preconsolidation stress determined from laboratory consolidation tests with the profile of effective over-burden pressures.

2.2.3.3 ESTIMATE PRECONSOLIDATION from c/P_c ratio, where c is the cohesion ($q_{u/2}$) and P_c is the preconsolidation stress, using laboratory data from unconfined compression test and Atterberg limits.

2.2.3.4 IF UNDERCONSOLIDATION IS INDICATED, install piezometers to measure the magnitude of hydrostatic excess pore water pressures.

2.2.4 COMPUTATION OF ADDED STRESSES. Use the elastic solutions to determine the vertical stress increment from applied loads. On vertical lines beneath selected points in the loaded area, plot profiles of estimated preconsolidation and effective overburden stress plus the increment of applied stress. See Figure 3 for typical profiles. Lowering of groundwater during construction or regional drawdown increases effective stress at the boundaries of the compressible stratum and initiates consolidation. Stress applied by drawdown equals the reduction in buoyancy of overburden corresponding to decrease in boundary water pressure. In developed locations, settlement of surrounding areas from drawdown must be carefully evaluated before undertaking dewatering or well pumping.

3. INSTANTANEOUS SETTLEMENT

3.1 IMMEDIATE SETTLEMENT OF FINE-GRAINED SOILS. Generally, the instantaneous settlement results from elastic compression of clayey soil. For foundations on unsaturated clay or highly overconsolidated clay, the elastic settlement constitutes a significant portion of the total settlement. Immediate settlement Δ_V is estimated as:

$$\Delta_V = q \times B \times [(1 - \gamma_2)/E_U] \times I$$

q is applied uniform pressure; B is width of loaded area; I is combined shape and rigidity factor; γ is Poisson's ratio - ranges between 0.3 and 0.5, the higher value being for saturated soil with no volume change during loading; and E_U is undrained modulus obtained from laboratory or field (pressuremeter) tests. Table 1 provides values of I (refer to Stresses and Deflections in Foundations and Pavements, by Department of Civil Engineering, University of California, Berkeley, CA). Empirical relationship derived from field measurement may be used to determine E_U when actual test values are not available; see Table 2 (refer to An Engineering Manual For Settlement Studies, by Duncan and Buchignani). Empirical correlations for estimation of OCR (Over Consolidation Ratio) are available in the technical literature. If the factor of safety against bearing failure is less than about 3, then the immediate settlement Δ_V is modified as follows:

$\Delta_C = \Delta_{SR}$, where:

Δ_C = immediate settlement corrected to allow for partial yield condition
SR = Settlement Ratio

Determine SR from Figure 4 (refer to Initial Settlement of Structures on Clay, by D'Appolonia, et al.). See Figure 5 for an example.

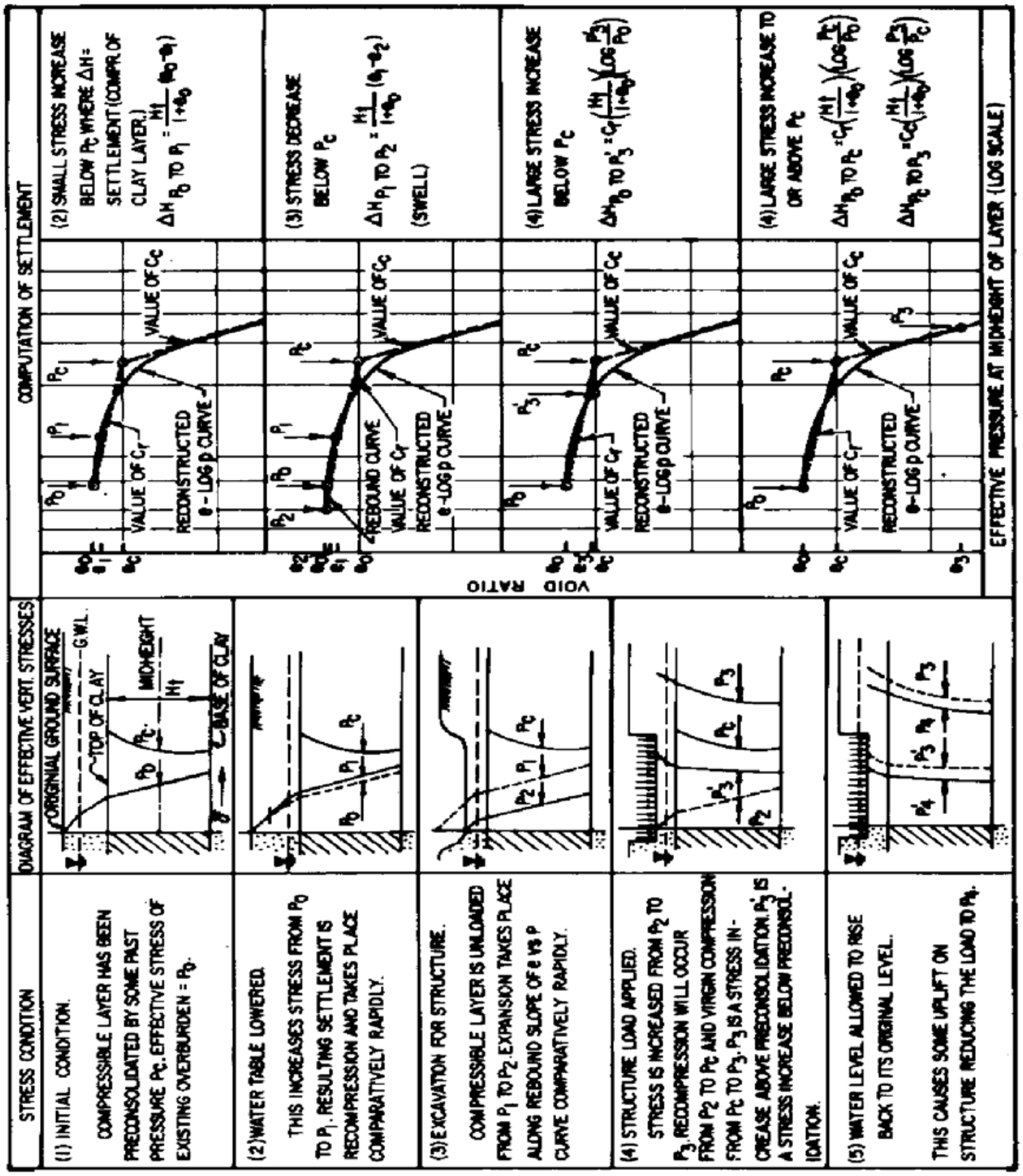


FIGURE 3
Computation of Total Settlement for Various Loading Conditions

Figure 3

Computation of Total Settlement for Various Loading Conditions

3.2. SETTLEMENT OF COARSE-GRAINED SOILS. This immediate settlement is a function of the width and depth of footing, elevation of the water table, and the modulus of vertical subgrade reaction (K_{VI}) within the depth affected by the footing. Figure 6 may be used to estimate K_{VI} from the soil boring log, and to compute anticipated settlement. For large footings where soil deformation properties vary significantly with depth or where the thickness of granular soil is only a fraction of the width of the loaded area, the method in Figure 6 may underestimate settlement.

3.3 TOTAL SETTLEMENT IN GRANULAR SOILS. Total settlement is the combined effect of immediate and long-term settlements. A usually conservative estimate of settlement can be made utilizing the method in Figure 7 (Refer to Static Cone to Compute Static Settlement Over Sand, by Schmertmann). A review of methods dealing with settlement of sands utilizing the standard penetration test results can be found in Equivalent Linear Model for Predicting Settlements of Sand Bases, by Oweis.

Shape and Rigidity Factor I for Loaded Areas on an Elastic Half-Space of Infinite Depth				
Shape and Rigidity	Center	Corner	Edge/Middle of Long Side	Average
Circle (flexible)	1.00		0.64	0.85
Circle (rigid)	0.79		0.79	0.79
Square (flexible)	1.12	0.56	0.76	0.95
Square (rigid)	0.85	0.82	0.82	0.82
Rectangle (flexible) Length/width				
2	1.53	0.76	1.12	1.30
5	2.10	1.05	1.68	1.62
10	2.56	1.28	2.10	2.04
Rectangle (rigid) Length/width				
2	1.12	1.12	1.12	1.12
5	1.60	1.60	1.60	1.60
10	2.00	2.00	2.00	2.00

Table 1

Shape and Rigidity Factors I for Calculating Settlements

of Points on Loaded Areas at the Surface of an Elastic Half-Space

TABLE 1 (continued)
Shape and Rigidity Factors I for Calculating Settlements
of Points on Loaded Areas at the Surface of an Elastic Half-Space

Shape and Rigidity Factor I for Loaded Areas on an Elastic Half-Space of Limited Depth Over a Rigid Base

H/B	Center of Rigid Circular Area Diameter = B	Corner of Flexible Rectangular Area				
		L/B = 1	L/B = 2	L/B = 5	L/B = 10	(strip) L/B = ∞
for ν = 0.50						
0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0.14	0.05	0.04	0.04	0.04	0.04
1.0	0.35	0.15	0.12	0.10	0.10	0.10
1.5	0.48	0.23	0.22	0.18	0.18	0.18
2.0	0.54	0.29	0.29	0.27	0.26	0.26
3.0	0.62	0.36	0.40	0.39	0.38	0.37
5.0	0.69	0.44	0.52	0.55	0.54	0.52
10.0	0.74	0.48	0.64	0.76	0.77	0.73
for ν = 0.33						
0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0.20	0.09	0.08	0.08	0.08	0.08
1.0	0.40	0.19	0.18	0.16	0.16	0.16
1.5	0.51	0.27	0.28	0.25	0.25	0.25
2.0	0.57	0.32	0.34	0.34	0.34	0.34
3.0	0.64	0.38	0.44	0.46	0.45	0.45
5.0	0.70	0.46	0.56	0.60	0.61	0.61
10.0	0.74	0.49	0.66	0.80	0.82	0.81

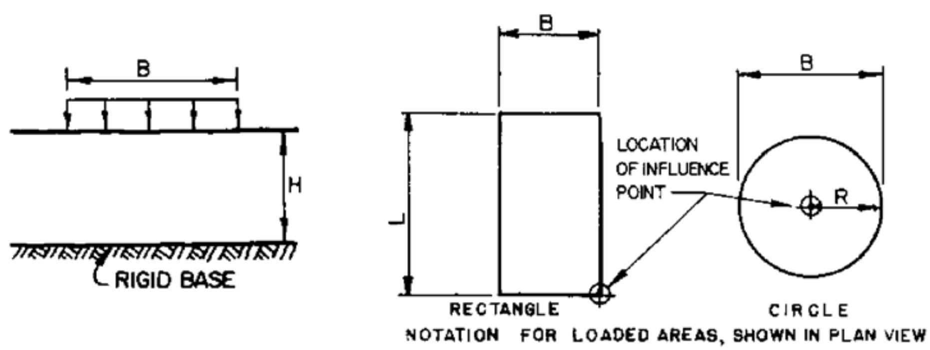


Table 1 (continued)

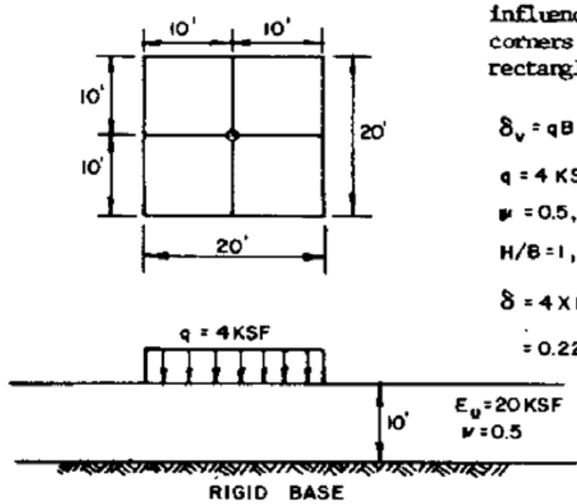
Shape and Rigidity Factors I for Calculating Settlements

of Points on Loaded Areas at the Surface of an Elastic Half-Space

TABLE 1 (continued)
Shape and Rigidity Factors I for Calculating Settlements
of Points on Loaded Areas at the Surface of an Elastic Half-Space

Example:

Compute immediate settlement at center of uniformly loaded area (flexible) measuring 20' by 20'.



Calculate as the sum of the influence values at the corners of four equal-sided rectangles.

$$\delta_v = qB\,\frac{1-\nu^2}{E_u}\,I$$

q = 4 KSF, B = 10'

ν = 0.5, Eu = 20 KSF

H/B = 1, L/B = 1 I = 0.15

$$\delta = 4 \times 10 \times \left[\frac{1-0.5^2}{20}\right] \times 0.15$$

$$= 0.225'$$

Table 1 (continued)

Shape and Rigidity Factors I for Calculating Settlements

of Points on Loaded Areas at the Surface of an Elastic Half-Space

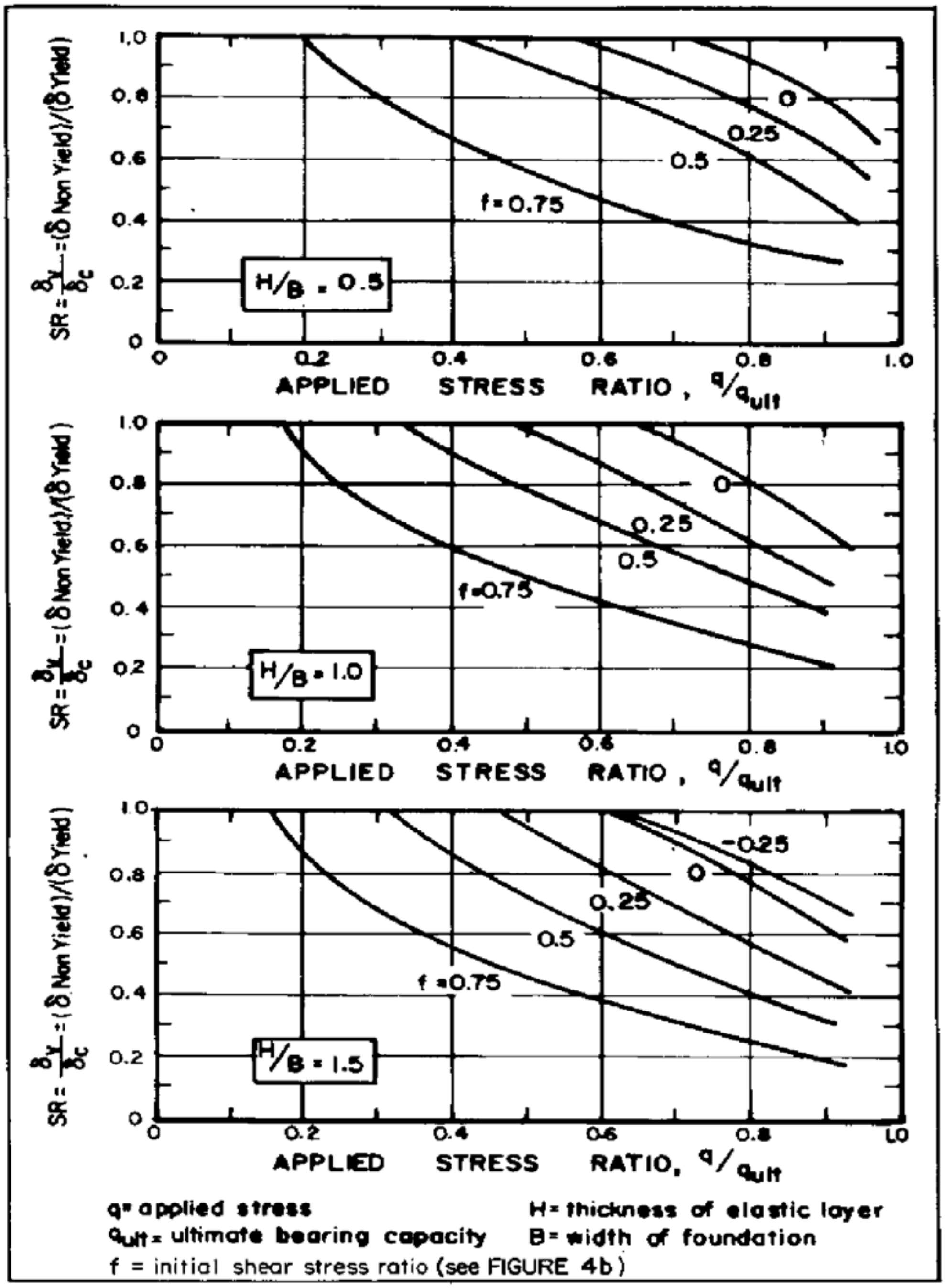


FIGURE 4a
Relationship Between Settlement Ratio and Applied Stress Ratio for Strip Foundation on Homogeneous Isotropic Layer

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Relationship Between Settlement Ratio and Applied Stress Ratio For Strip Foundation on Homogeneous Isotropic Layer

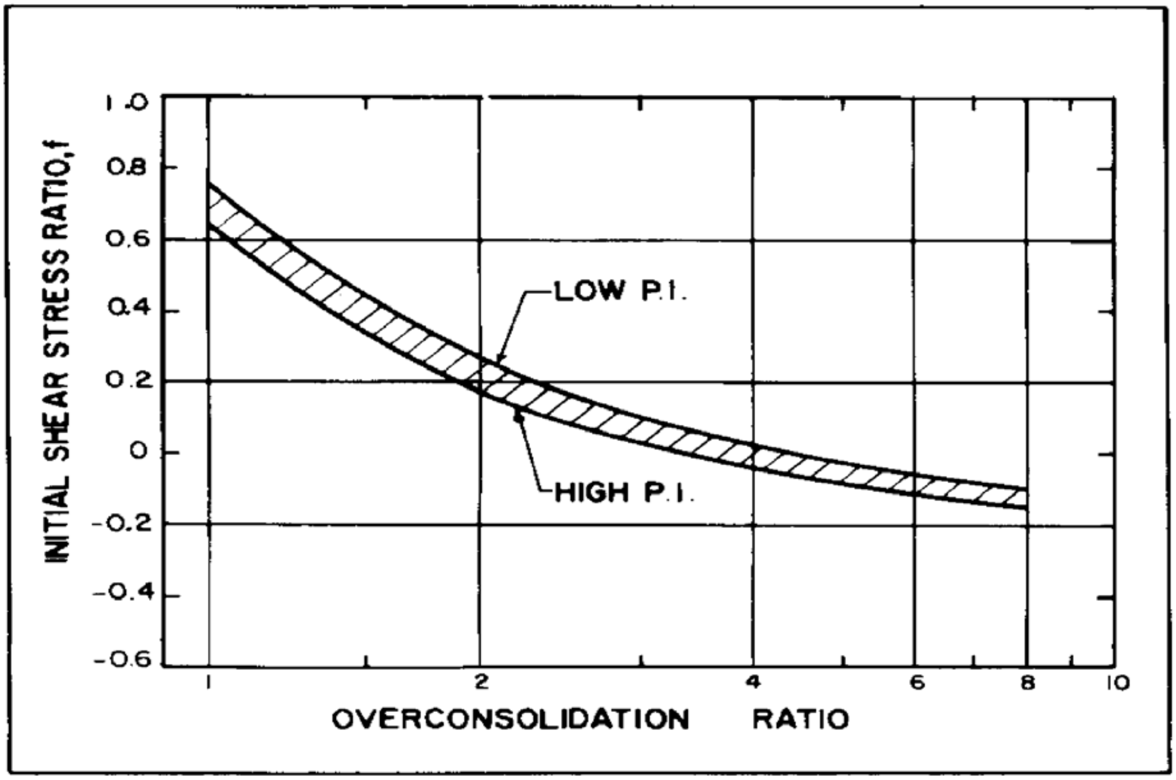


FIGURE 4b
Relationship Between Initial Shear Stress and Overconsolidation Ratio

Figure 4b

Relationship Between Initial Shear Stress

And Overconsolidation Ratio

Example:

Given LL = 58% PI = 25% c = 1 KSF
Moderately consolidated clay, OCR <3
Depth to rigid layer (H) = 10.5 ft
$\gamma = 0.5$
Rigid strip footing, width = 7 ft   q+appl, = 2.5 KSF   q+ult, = 6 KSF

Find immediate settlement.

$\Delta_V = q \times B \times [(1 - \gamma^2)/E_U] \times I$

I = 2.0 (Table 1) assume length/width [approximately] 10

From Table 2, $E_U = 600$

$E_U = 600 \times 1 = 600$ KSF

$\Delta_V = 2.5 \times 7 \times [(1 - 0.5^2)/600] \times 2.0 \times 12 = 0.52$ inches

Find factor of safety against bearing failure.

$F_S = 6.0/2.5 = 2.4 < 3.0$

Correct for yield.

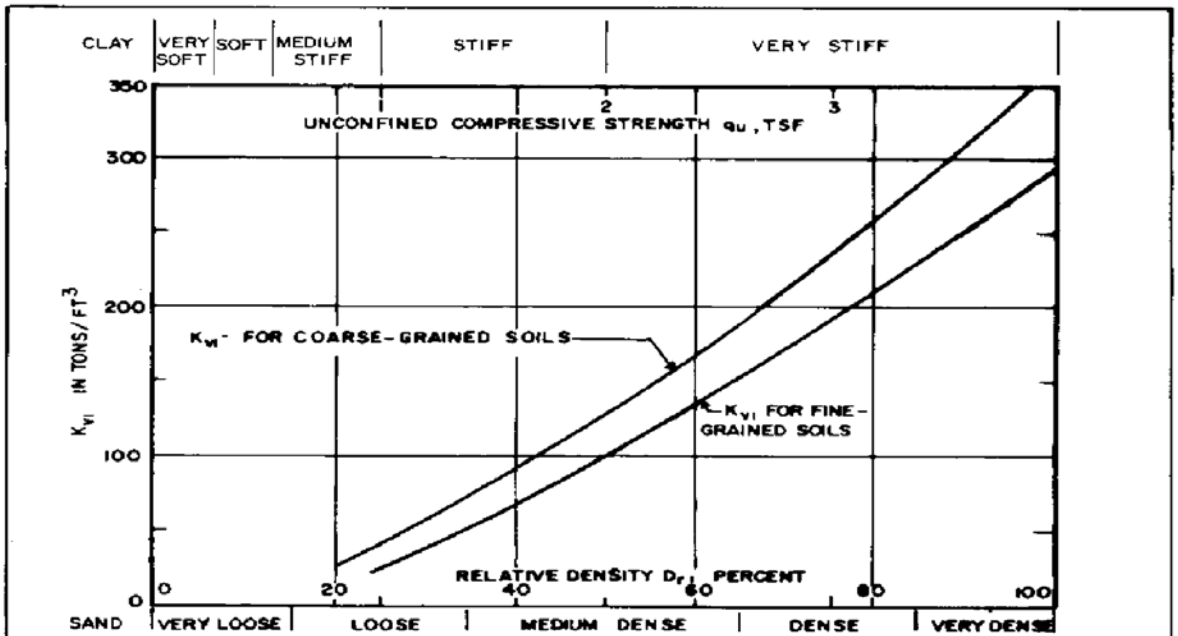
f = 0.7 (Figure 4b)

$q_{appl} / q_{ult} = 0.42$, H/B = 1.5

SR = 0.60 (Figure 4a)

Figure 5

Example of Immediate Settlement Computations in Clay



DEFINITIONS

ΔH_i = IMMEDIATE SETTLEMENT OF FOOTING
q = FOOTING UNIT LOAD IN tsf
B = FOOTING WIDTH
D = DEPTH OF FOOTING BELOW GROUND SURFACE
K_{vi} = MODULUS OF VERTICAL SUBGRADE REACTION

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)

SHALLOW FOOTINGS $D \leq B$

FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{4\, q\, B^2}{K_{vi}\,(B+1)^2}$$

FOR $B \geq 40$ FT:

$$\Delta H_i = \frac{2\, q\, B^2}{K_{vi}\,(B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{2\, q\, B^2}{K_{vi}\,(B+1)^2}$$

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.
2. VALUES OF K_{vi} SHOWN FOR COARSE-GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUNDWATER LEVEL AT A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING. IF GROUNDWATER IS AT BASE OF FOOTING, USE $K_{vi}/2$ IN COMPUTING SETTLEMENT
3. FOR CONTINUOUS FOOTINGS MULTIPLY THE SETTLEMENT COMPUTED FOR WIDTH "B" BY 2.

FIGURE 6
Instantaneous Settlement of Isolated Footings on Coarse-Grained Soils

Figure 6

Instantaneous Settlement of Isolated Footings on Coarse-Grained Soils

DATA REQUIRED:

1. A profile of standard penetration resistance N (blows/ft) versus depth, from the proposed foundation level to a depth of 2B, or to boundary of an incompressible layer, whichever occurs first. Value of soil modulus E_S is established using the following relationships.

Soil Type	E_S/N
Silts, sands silts, slightly cohesive silt-sand mixtures	4
Clean, fine to med, sands & slightly silty sands	7
Coarse sands & sands with little gravel	10
Sandy gravels and gravel	12

2. Least width of foundation = B, depth of embedment = D, and proposed average contact pressure = P.

3. Approximate unit weights of surcharge soils, and position of water table if within D.

4. If the static cone bearing value q_C measured compute E_S based on $E_S = 2\ q_C$.

ANALYSIS PROCEDURE:

Refer to table in example problem for column numbers referred to by parenthesis:

1. Divide the subsurface soil profile into a convenient number of layers of any thickness, each with constant N over the depth interval 0 to 2B below the foundation.

2. Prepare a table as illustrated in the example problem, using the indicated column headings. Fill in columns 1, 2, 3 and 4 with the layering assigned in Step 1.

3. Multiply N values in column 3 by the appropriate factor E_S/N (col. 4) to obtain values of E_S; place values in column 5.

4. Draw an assumed 2B-0.6 triangular distribution for the strain influence factor I_Z along a scaled depth of 0 to 2B below the foundation. Locate the depth of the mid-height of each of the layers assumed in Step 2, and place in column 6. From this construction, determine the I_Z value at the mid-height of each layer, and place in column 7.

Figure 7

Settlement of Footings Over Granular Soils: Example Computation

Using Schmertmann's Method

5. Calculate $(I_z/E_s)\ \Delta Z$, and place in column 8. Determine the sum of all values in column 8.

6. Total settlement $= \Delta H = C_1 C_2 \Delta p \sum_{0}^{2B} \left(\frac{I_z}{E_s}\right) \Delta Z$,

where $C_1 = 1 - 0.5\ (p_o/\Delta p)$; $C_1 \geq 0.5$ embedment correction factor

$C_2 = 1 + 0.2 \log (10t)$ creep correction factor

p_o = overburden pressure at foundation level

Δp = net foundation pressure increase

t = elaspsed time in years.

EXAMPLE PROBLEM:

GIVEN THE FOLLOWING SOIL SYSTEM AND CORRESPONDING STANDARD PENETRATION TEST (SPT) DATA, DETERMINE THE AMOUNT OF ULTIMATE SETTLEMENT UNDER A GIVEN FOOTING AND FOOTING LOAD:

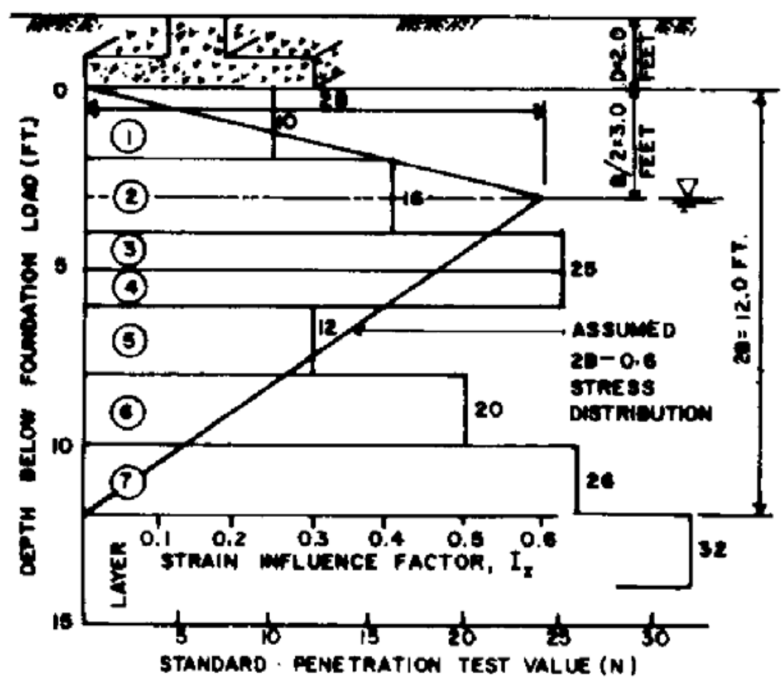


FIGURE 7 (continued)
Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

Figure 7 (continued)
Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

Footing Details:

Footing width: 6.0 ft. (min.) by 8.0 ft. (max.)

Depth of Embedment: 2.0 ft. Load (Dead + Live): 120 tons

Soil Properties:

Depth Below Surface (ft.)	Depth Below Base of Footing (ft.)	Unit Wt. (pcf) Moist	Sat.	Soil Description
0 - 5	<5	95	105	Fine sandy silt
5 - 10	3 - 8	105	120	Fine to medium sand
10 - 17	8 - 15	120	130	Coarse sand

Solution:

Layer (1)	ΔZ (in.) (2)	N (3)	E_s/N (4)	E_s (tsf) (5)	Z_c (in.) (6)	I_z (7)	$\frac{I_z}{E_s}\Delta Z$ (in./tsf) (8)
1	24	10	4	40	12	.20	0.120
2	24	16	4	64	36	.60	0.225
3	12	25	4	100	54	.50	0.060
4	12	25	7	175	66	.43	0.029
5	24	12	7	84	84	.33	0.094
6	24	20	7	140	108	.20	0.034
7	24	26	10	260	132	.07	0.006

$\Sigma = 0.568$

$p_o = (2.0 \text{ ft})(95 \text{ pcf}) = 190 \text{ psf} = 0.095 \text{ tsf}$

$\Delta p = 120 \text{ tons}/(6 \text{ ft.})(8 \text{ ft.}) = 2.50 \text{ tsf}$

At t = 1 yr,

$$C_1 = 1 - 0.5(.095/2.50) = 0.981$$

$$C_2 = 1 + 0.2 \log (10)(1) = 1.20$$

$$\Delta H = (0.981)(1.20)(2.50)(0.568) = \underline{1.67 \text{ in.}}$$

FIGURE 7 (continued)
Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

Figure 7 (continued)

Settlement of Footings Over Granular Soils:

Example Computation Using Schmertmann's Method

4. PRIMARY AND SECONDARY SETTLEMENTS.

4.1 PRIMARY CONSOLIDATION.

4.1.1 CONSOLIDATION SETTLEMENT. For conditions where excess pore pressures are developed during the application of load and if preconsolidation stress is determined reliably, total settlement can be predicted with reasonable accuracy. The percentage error is greatest for settlement from recompression only. In this case an overestimate may result unless high quality undisturbed samples are used for consolidation tests.

4.1.1.1. TYPICAL LOADING CYCLE. See Figure 3 for loading sequence in building construction. Foundation excavation can cause swell and heave. Application of a structural load recompresses subsoil and may extend consolidation into the virgin range. Stress changes are plotted on a semi-logarithmic pressure-void ratio e-log p curve similar to that shown in Figure 3.

4.1.1.2 PRESSURE-VOID RATIO DIAGRAM. Determine the appropriate e-log p curve to represent average properties of compressible stratum from consolidation tests. The e-log p curve may be interpreted from straight line virgin compression and recompression slopes intersecting at the preconsolidation stress. Draw e-log p curve to conform to these straight lines as shown in Figure 3.

4.1.1.3 MAGNITUDE OF CONSOLIDATION SETTLEMENT. Compute settlement magnitude from change in void ratio corresponding to change in stress from initial to final conditions, obtained from the e-log p curve (Figure 3). To improve the accuracy of computations divide the clay layer into a number of sublayers for computing settlement. Changes in compressibility of the stratum and existing and applied stresses can be dealt with more accurately by considering each sublayer independently and then finding their combined effect.

4.1.1.4 PRELIMINARY ESTIMATES of C_C can be made using the correlations in Table 3.

4.1.2 CORRECTIONS TO MAGNITUDE OF CONSOLIDATION SETTLEMENTS. Settlements computed for overconsolidated clays by the above procedures may give an overestimate of the settlement. Correct consolidation settlement estimate as follows:

$H_C = \alpha\ (W - \Delta H)_{OC}$

H_C = corrected consolidation settlement

α = function of overconsolidation ratio (OCR)

OCR = preconsolidation pressure/overburden pressure (P_C/P_O)

$([W\text{-}\Delta]H)_{OC}$ = calculated settlement resulting from stress increment of P_O to P_C

For the width of loaded area and thickness of compressible stratum, see Figure 8 for values and refer to Estimating Consolidation Settlements of Shallow Foundation on Overconsolidated Clay, by Leonards.

$C_C = 0.009\ (LL - 10\%)$ inorganic soils, with sensitivity less than 4

$C_C = 0.0115\ w_n$ organic soils, peat *

$C_C = 1.15\ (e{+}o, - 0.35)$ all clays *

$C_C = (1 + e_0)(0.1 + [w_n - 25]\ 0.006)$ varved clays *

w_n is natural moisture content, LL is water content at liquid limit and e_0 is initial void ratio.

Table 3

Estimates of Coefficient of Consolidation (C_C)

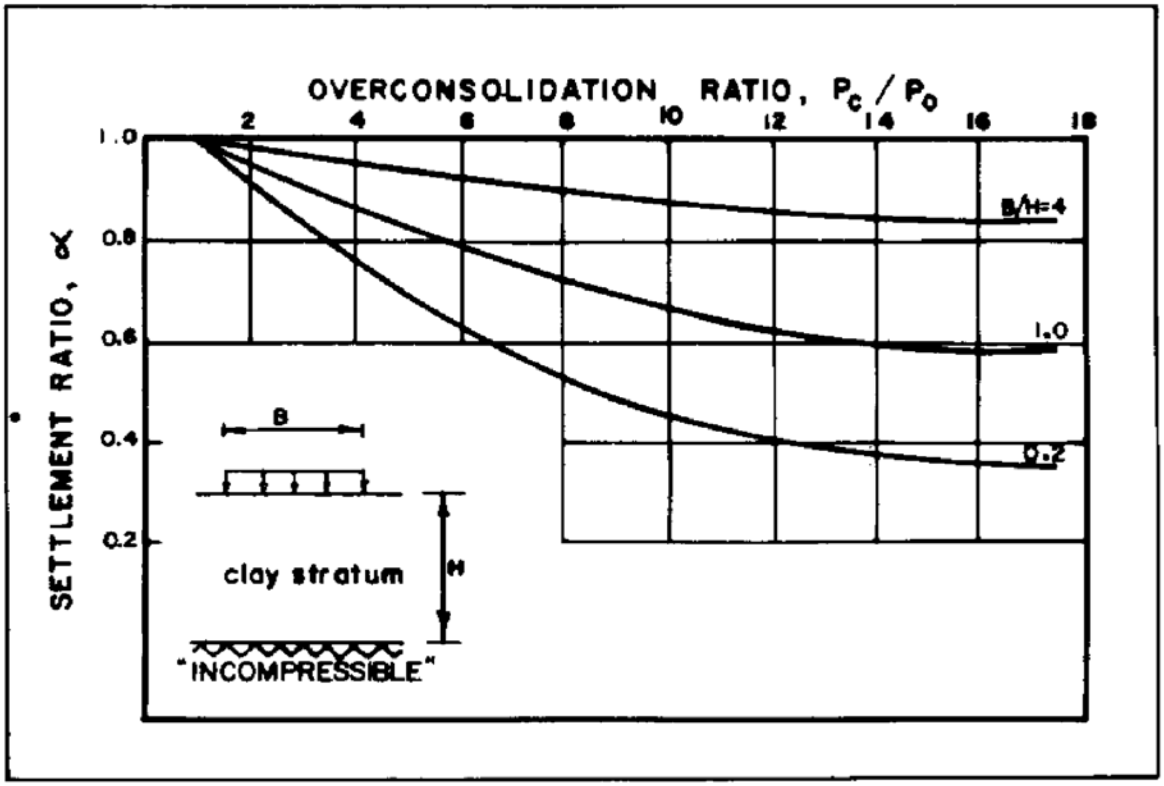


FIGURE 8
Relation Between Settlement Ratio and Overconsolidation Ratio

Figure 8

Relation Between Settlement Ratio and Overconsolidation Ratio

4.2 TIME RATE OF PRIMARY CONSOLIDATION.

4.2.1 APPLICATION. Settlement time rate must be determined for foundation treatment involving either acceleration of consolidation or preconsolidation before construction of structure. Knowledge of settlement rate or percent consolidation completed at a particular time is important in planning remedial measures on a structure damaged by settlement.

4.2.2 TIME RATE OF CONSOLIDATION. Where pore water drainage is essentially vertical, the ordinary one dimensional theory of consolidation defines the time rate of settlement. Using the coefficient of consolidation c_V compute percent consolidation completed at specific elapsed times by the time factor T_V curves of Figure 9 (upper panel), refer to, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures, DOD. For vertical sand drains use Figure 10 (upper panel, same reference). For preliminary estimates, the empirical correlation for c_V may be used.

4.2.2.1 EFFECT OF PRESSURE DISTRIBUTION. Rate of consolidation is influenced by the distribution of the pressures which occur throughout the depth of the compressible layer. For cases where the pressures are uniform or vary linearly with depth, use Figure 9 which includes the most common pressure distribution. The nomograph in Figure 11 may be used for this case. For nonlinear pressure distribution, refer to Soil Mechanics in Engineering Practice, by Terzaghi and Peck, to obtain the time factor.

4.2.2.2 ACCURACY OF PREDICTION. Frequently the predicted settlement time is longer than that observed in the field for the following reasons:

4.2.2.2.1 THEORETICAL CONDITIONS ASSUMED for the consolidation analysis frequently do not hold in situ because of intermediate lateral drainage, anisotropy in permeability, time dependency of real loading, and the variation of soil properties with effective stress. Two or three dimensional loading increases the time rate of consolidation. Figure 12 gives examples of how the width of the loaded area and anistropy in permeability can affect the consolidation rate substantially. As the ratio of the thickness of the compressible layer to the width of the loaded area increases, the theory tends to overestimate the time factor. For deposits such as some horizontal varved clays where continuous seams of high permeability are present, consolidation can be expected to be considerably faster than settlement rates computed based on the assumption of no lateral drainage.

4.2.2.2.2 THE COEFFICIENT OF CONSOLIDATION, as determined in the laboratory, decreases with sample disturbance. Predicted settlement time tends to be greater than actual time.

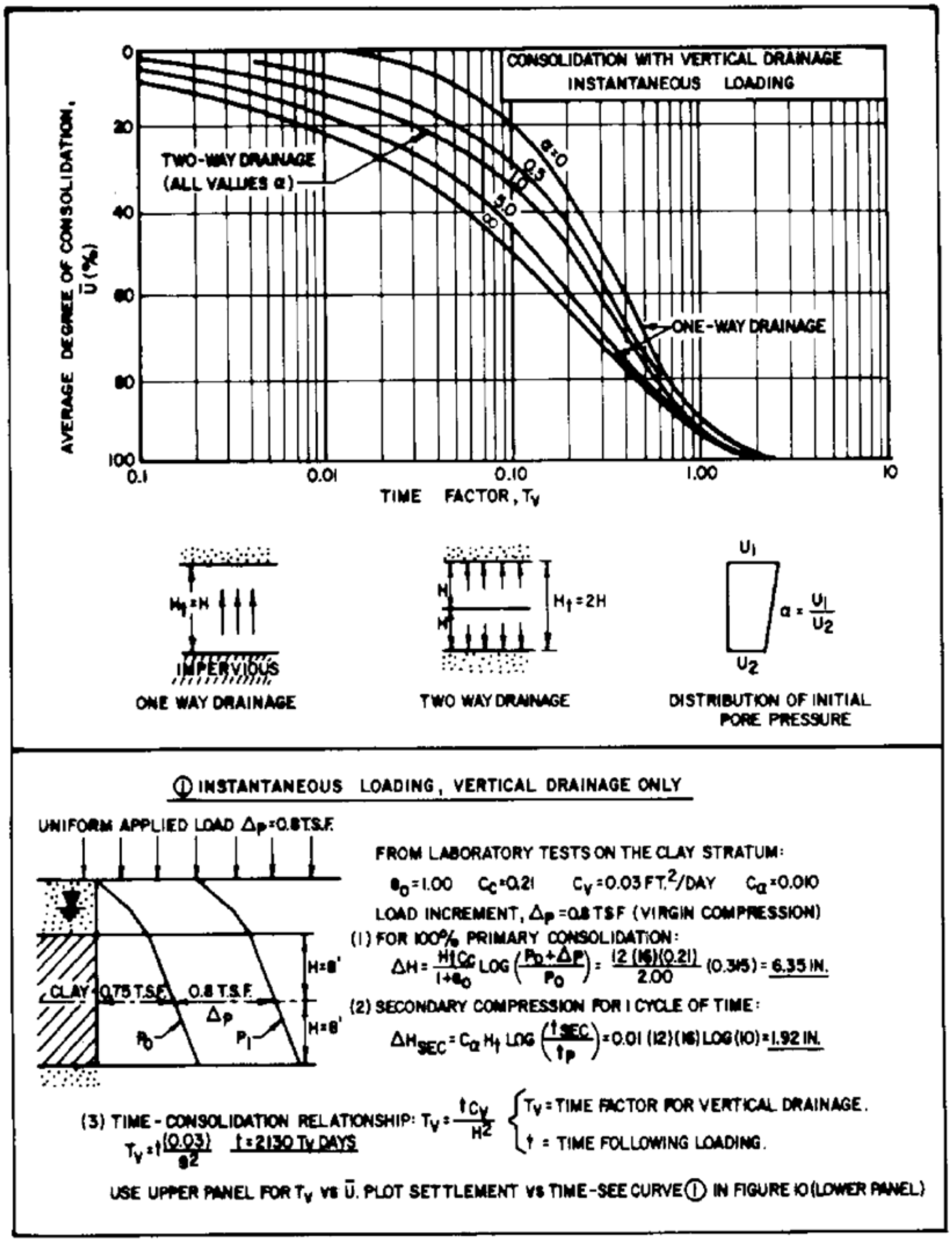


FIGURE 9
Time Rate of Consolidation for Vertical Drainage
Due to Instantaneous Loading

Figure 9

Time Rate of Consolidation for Vertical Drainage

Due to Instantaneous Loading

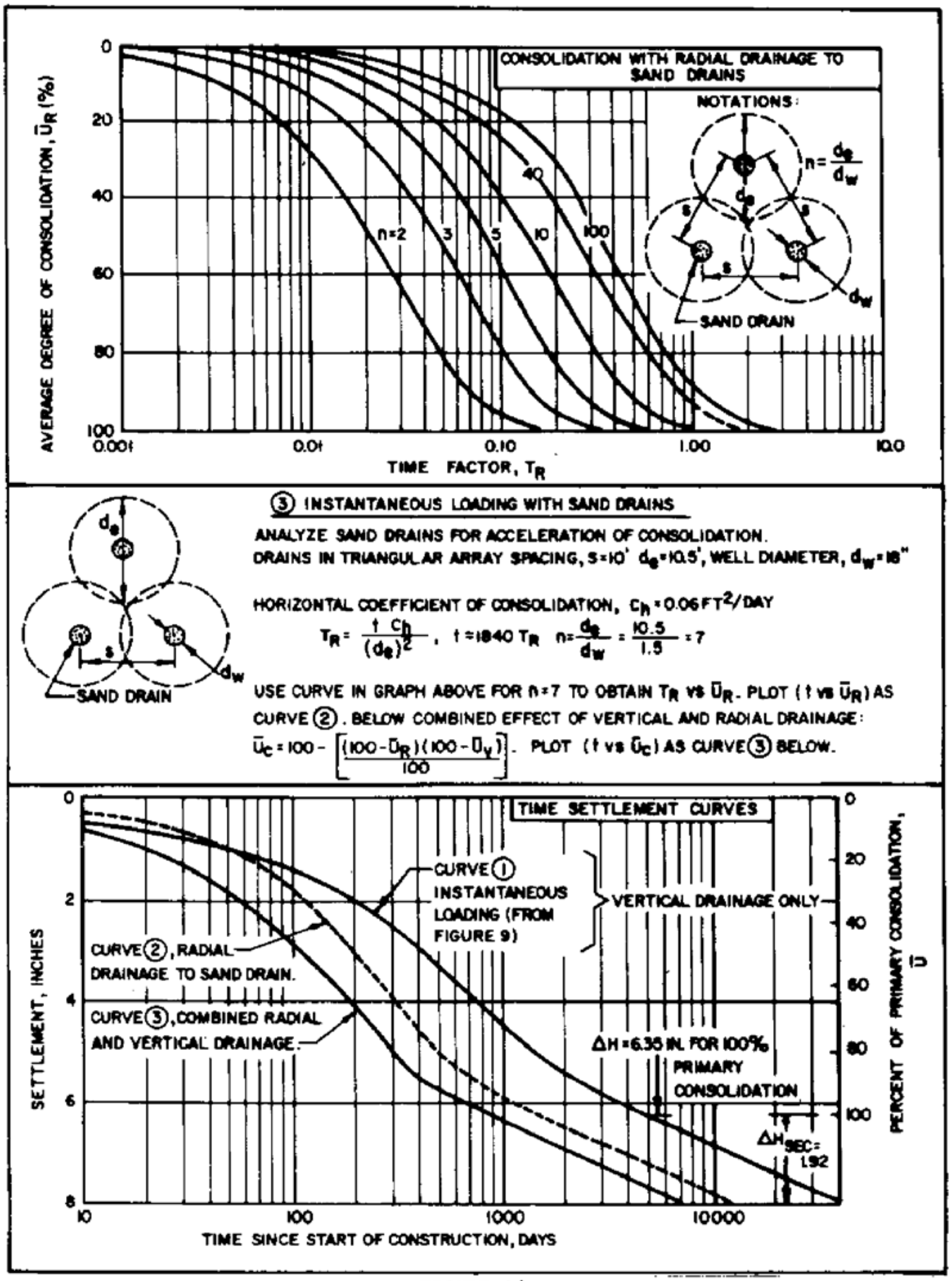


FIGURE 10
Vertical Sand Drains and Settlement Time Rate
7.1-228

Figure 10

Vertical Sand Drains and Settlement Time Rate

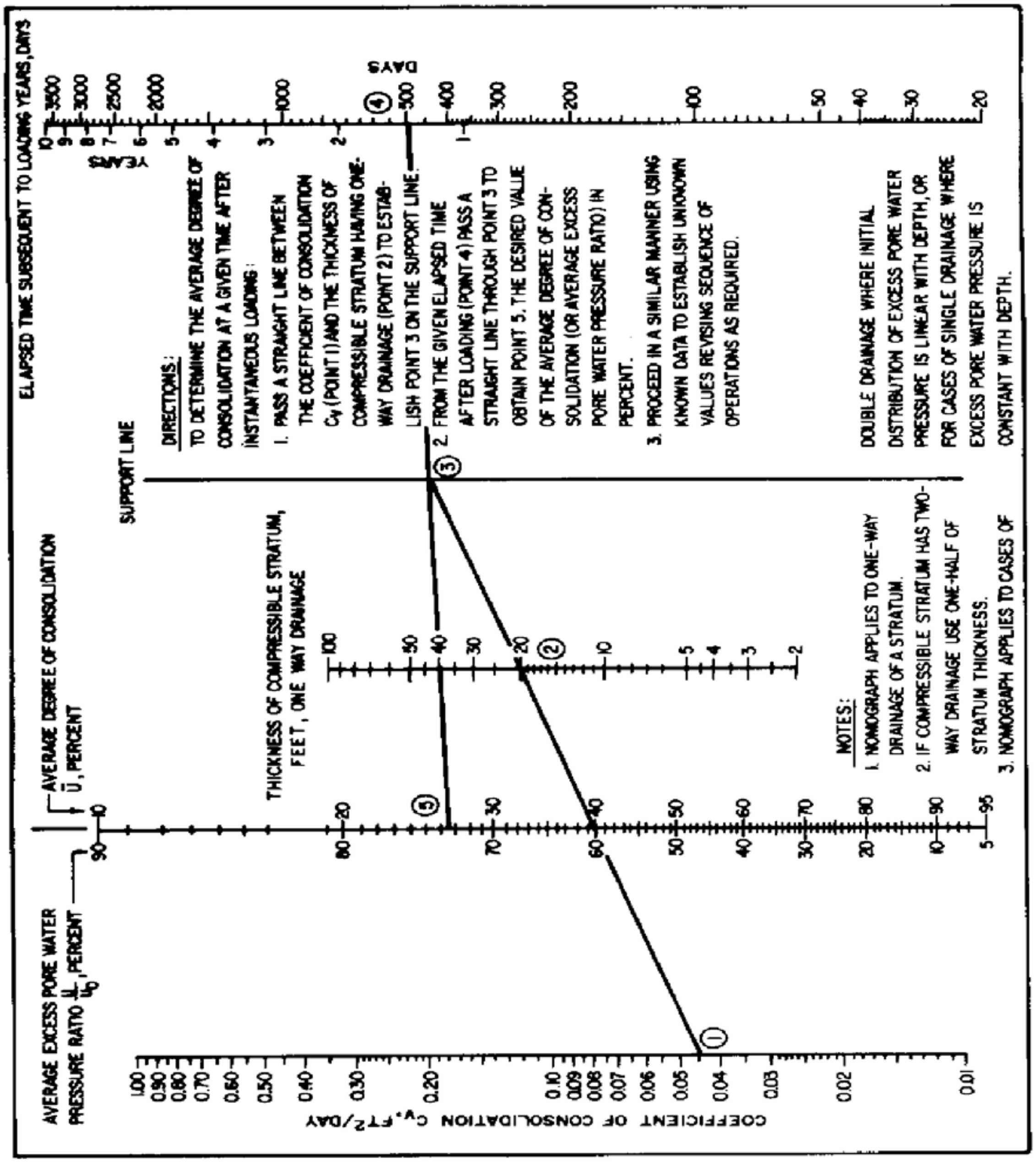


FIGURE 11
Nomograph for Consolidation With Vertical Drainage

Figure 11

Nomograph for Consolidation with Vertical Drainage

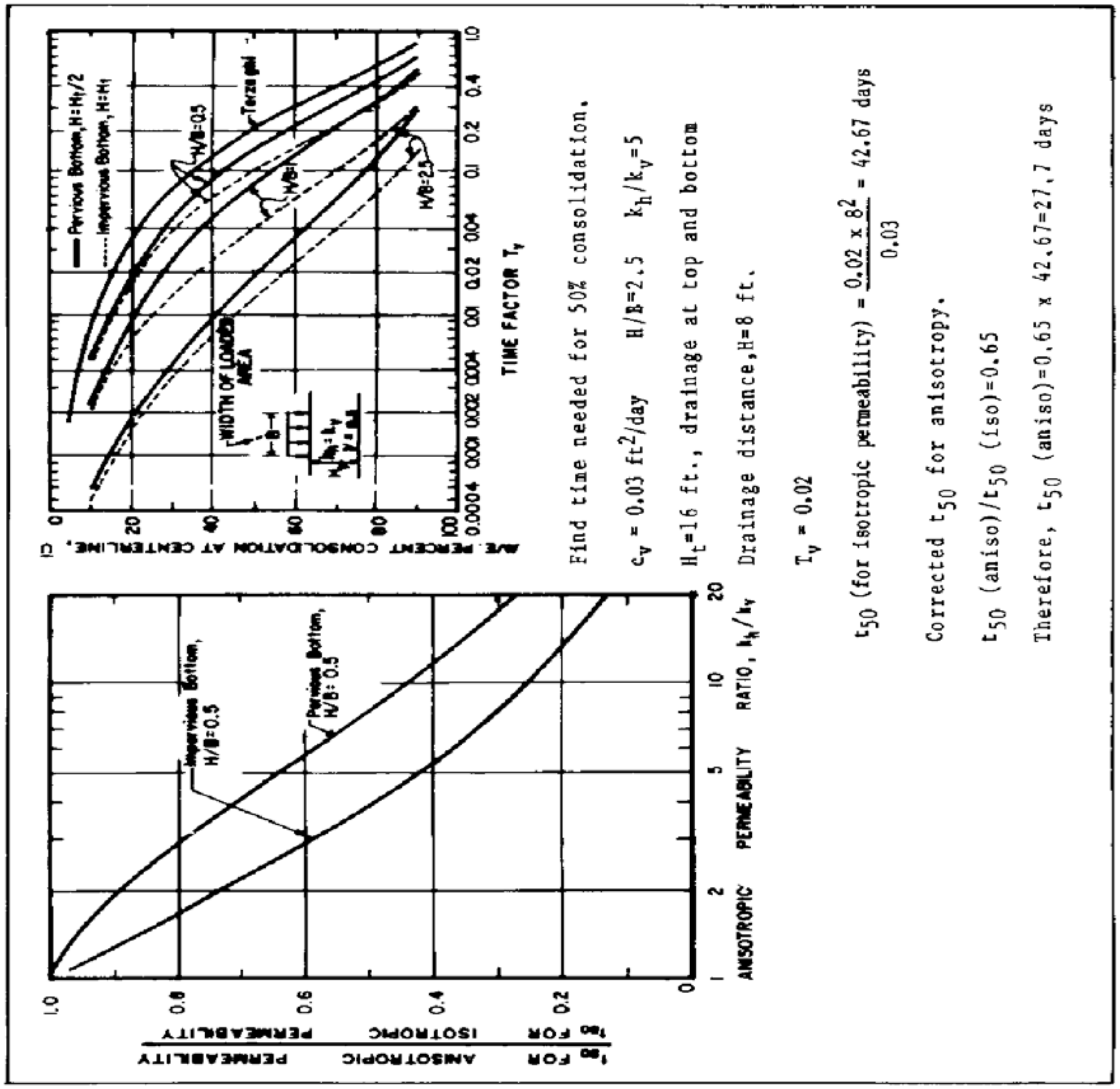


Figure 12

Effect of Drainage Conditions on Time Rate of Consolidation

4.2.2.2.3 GRADUAL LOAD APPLICATION. If construction time is appreciable compared to time required for primary consolidation, use the time factors of Figure 13 to determine consolidation rate during and following construction.

4.2.2.2.4 COEFFICIENT OF CONSOLIDATION FROM FIELD MEASUREMENTS. Where piezometers are installed to measure pore water pressure under the applied loads, c_V is computed as shown in Figure 14.

4.2.3 TIME RATE OF MULTI-LAYER CONSOLIDATION. If a compressible stratum contains layers of different overall properties, use the procedure of Figure 15 to determine overall settlement time rate.

4.3 SECONDARY COMPRESSION.

4.3.1 LABORATORY e-LOG p CURVE. A laboratory e-log p curve includes an amount of secondary compression that depends on duration of test loads. Secondary compression continues exponentially with time without definite termination. Thus, total or ultimate settlement includes secondary compression to a specific time following completion of primary consolidation.

4.3.2 SETTLEMENT COMPUTATION. Compute settlement from secondary compression following primary consolidation as follows:

$$H_{sec} = C_\alpha (H_t) (\log [t_{sec}/t_P]$$

where:

H_{sec} = settlement from secondary compression

C_α = coefficient of secondary compression expressed by the strain per log cycle of time

H_t = thickness of the compressible stratum

T_{sec} = useful life of structure or time for which settlement is significant

T_p = time of completion of primary consolidation

See example in Figure 9 for calculating the secondary settlement. The parameter C can be determined from laboratory consolidation tests; for preliminary estimates, the correlations in Figure 16 may be used. This relationship is applicable to a wide range of soils such as inorganic plastic clays, organic silts, peats, etc.

4.3.3 COMBINING SECONDARY AND PRIMARY CONSOLIDATION. If secondary compression is important, compute the settlement from primary consolidation separately, using an e-log p curve that includes only compression from primary consolidation. For each load increment in the consolidation test, compression is plotted versus time (log scale). The compression at the end of the primary portion (rather than 24 hours) may be used to establish e-log p curve.

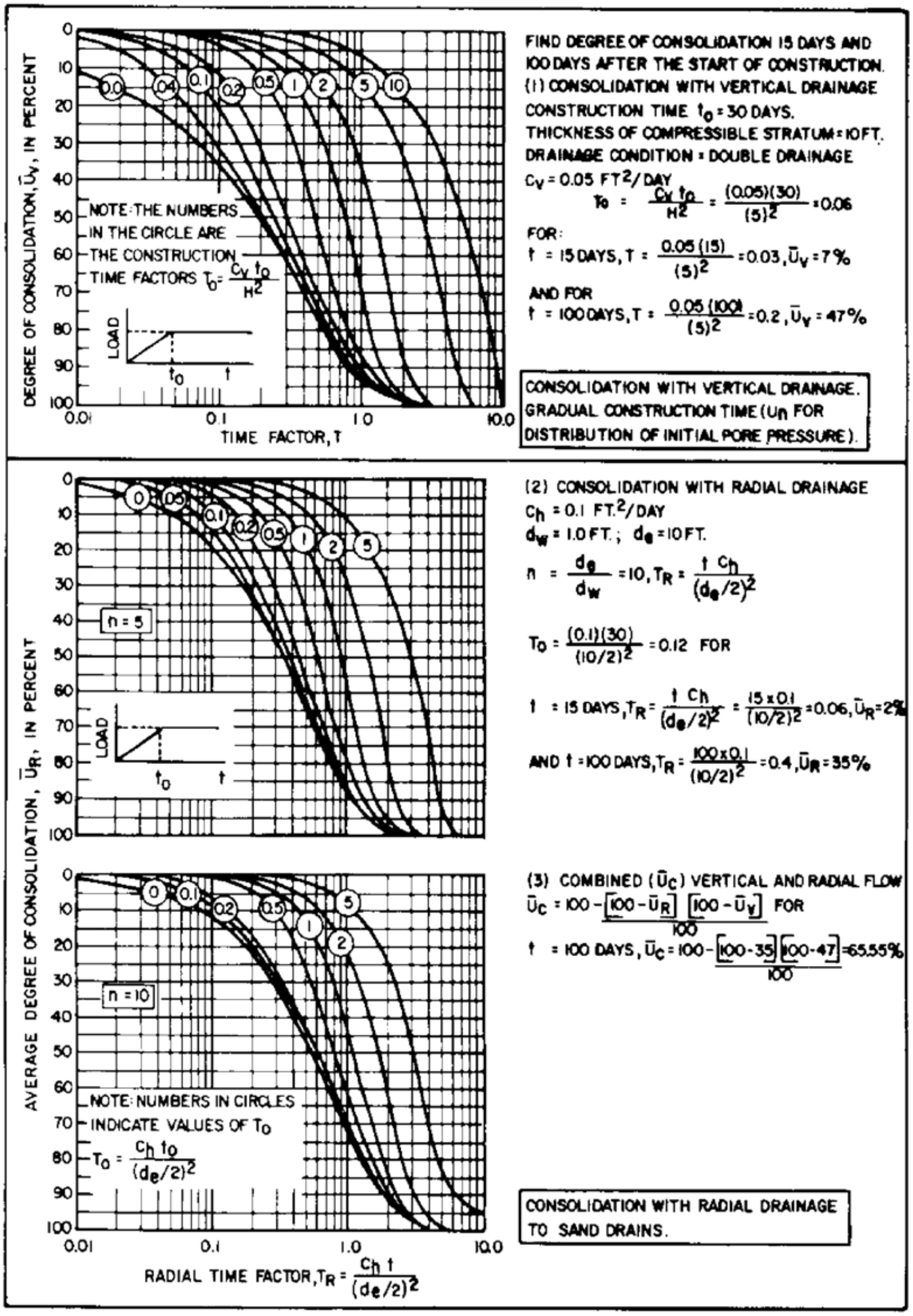
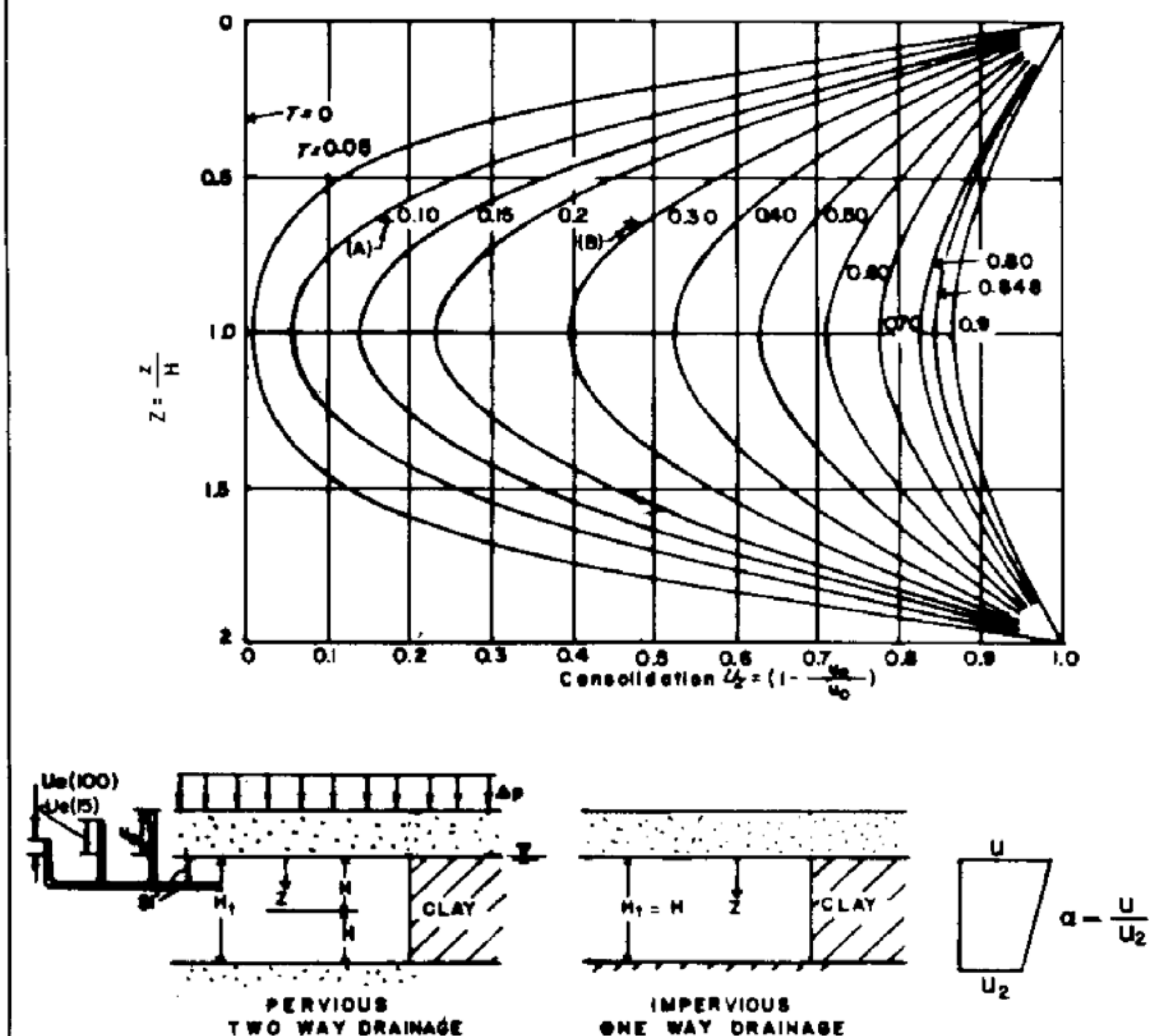


FIGURE 13
Time Rate of Consolidation for Gradual Load Application

Figure 13

Time Rate of Consolidation for Gradual Load Application



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

u_e = Excess pore pressure at some time t

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

FIGURE 14
Coefficient of Consolidation from Field Measurements

Figure 14

Coefficient of Consolidation from Field Measurements

Example:

Thickness of clay layer H_t = 66 ft, Drainage - top & bottom

H = 66/2 = 33 ft

Depth of piezometer below top of compressible layer = 21 ft

Applied external load [W-Δ]p = 1.5 KSF

Initial excess pore water pressure = u_o = [W-Δ]p = 1.5 KSF

Excess pore pressure after time t_1 = 15 days, $u_e(15)$ = 20 ft = U_{et1}

Excess pore pressure after time t_2 = 100 days, $u_e(100)$ = 14 ft = U_{et2}

Piezometer measure U_O = 24 feet of water +21 ft (initial static head) for a total of 45 ft.

Z/H = 0.21/0.33 = 0.64

Consolidation ratio at time t_1 = 15 days = $(u_Z)t_1$ = 1 - 20/24 = 0.17

Consolidation ratio at time t_2 = 100 days = $(u_2)t_2$ = 1 - 14/24 = 0.47

From above graph T_1 = 0.11 (point A), T_2 = 0.29 (point B)

$C_V = [(0.29 - 0.11)/(100-15)] \times (33)^2 = 231$ ft^2/day

FIGURE 14 (continued)

Coefficient of Consolidation from Field Measurements

For a soil system containing n layers with properties C_{vi} (coefficient of consolidation) and H_i (layer thickness), convert the system to one equivalent layer with equivalent properties, using the following procedure:

1. Select any layer i, with properties $c_v = c_{vi}$, $H = H_i$

2. Transform the thickness of every other layer to an equivalent thickness of a layer possessing the soil properties of layer i, as follows:

$$H'_1 = [(H_1)(c_{vi})^{1/2}]/c_{vi}$$

$$H'_2 = [(H_2)(c_{v2})^{1/2}]/c_{v2}$$

$$H'_n = [(H_n)(c_{vn})^{1/2}]/c_{vn}$$

3. Calculate the total thickness of the equivalent layer:

$$H'_T = H'_1 + H'_2 + \ldots + H'_i + \ldots + H'_n$$

4. Treat the system as a single layer of thickness H'_T, possessing a coefficient of consolidation $c_v = c_{vi}$

5. Determine values of percent consolidation (U) at various times (t) for total thickness (H'_T) using nomograph in Figure 11.

Figure 15

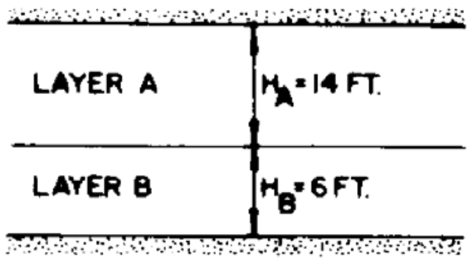
Procedure for Determining the Rate of Consolidation

for All Soil Systems Containing "N" layers

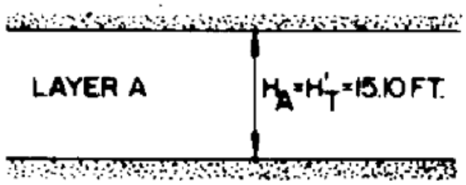
EXAMPLE OF COMPUTATION OF RATE OF CONSOLIDATION FOR A MULTI-LAYERED SYSTEM:

LAYERED SYSTEM:

ACTUAL STRATIFICATION



EQUIVALENT STRATIFICATION



KNOWN APPLICABLE SOIL PROPERTIES:

LAYER A:

$cv_A = 0.04\ FT.^2/DAY$

LAYER B:

$cv_B = 1.20\ FT.^2/DAY$

ASSUME: DOUBLE DRAINAGE

DETERMINATION OF EQUIVALENT LAYER THICKNESS:

1. ASSUME AN EQUIVALENT LAYER POSSESSING THE PROPERTIES OF SOIL A.
2. EQUIVALENT THICKNESS

$$\begin{aligned} H'_T &= H_A + H'_B \\ &= H_A + H_B \left(\frac{cv_A}{cv_B}\right)^{1/2} \\ &= 14 + 6 \left(\frac{0.04}{1.20}\right)^{1/2} \\ &= 14 + 1.10 \\ H'_T &= 15.10 \text{ FT.} \end{aligned}$$

3. DETERMINE $\bar{U}$ FROM FIGURE 11, e.g. AT 1 = 0.25 YEARS, USING H = (15.10)/2 = 7.55 FT. (DRAINAGE PATH ASSUMING DOUBLE DRAINAGE) AND $cv_A = 0.04\ FT.^2/DAY$, $\bar{U} = 29.1\%$

FIGURE 15 (continued)
Procedure for Determining the Rate of Consolidation
for All Soil Systems Containing "N" Layers

Figure 15 (continued)

Procedure for Determining the Rate of Consolidation

for All Soil Systems Containing “N” Layers

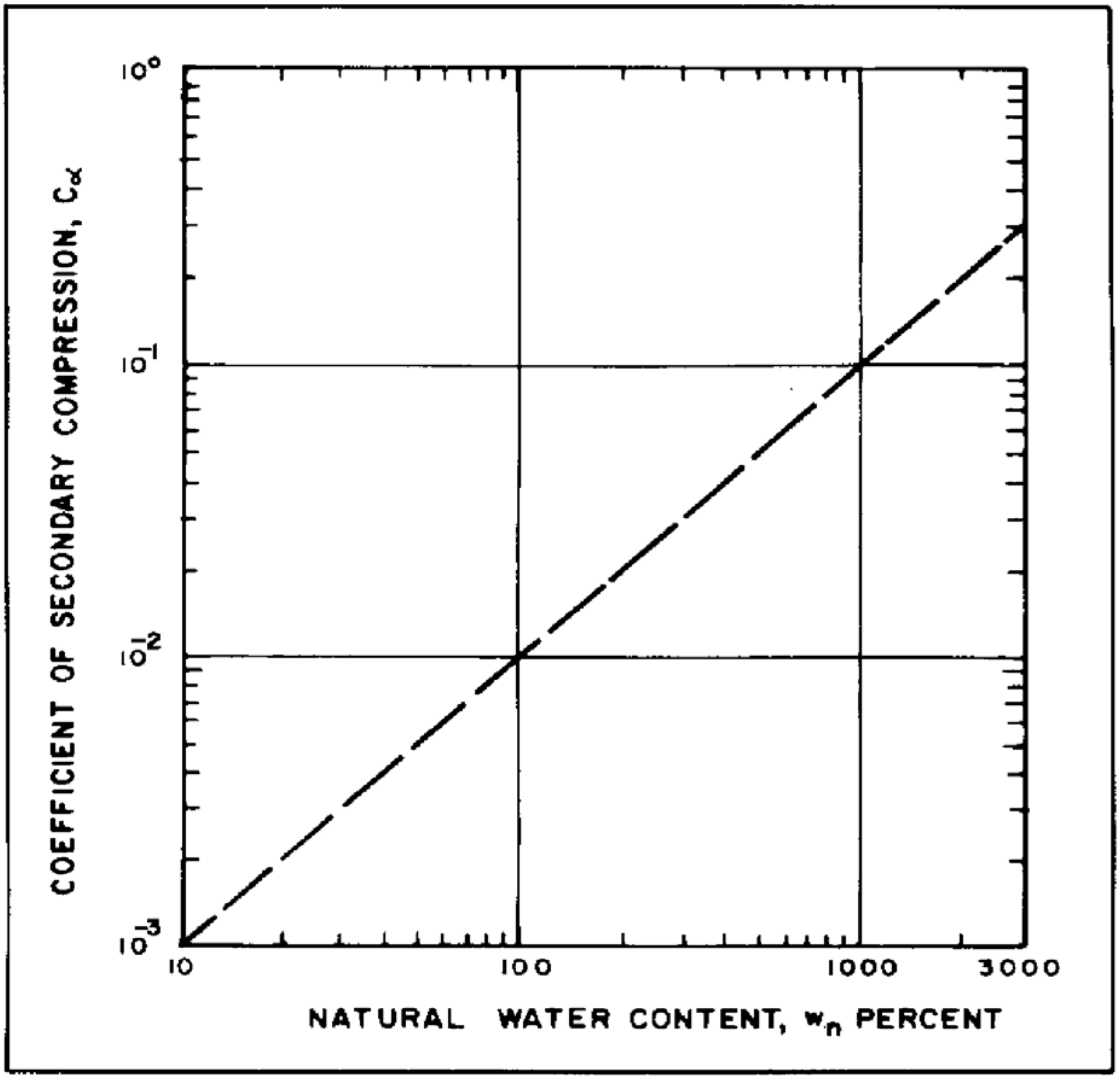


FIGURE 16
Coefficient of Secondary Compression as Related to Natural Water Content

Figure 16

Coefficient of Secondary Compression as Related to Natural Water Content

4.4 SANITARY LANDFILL. Foundations on sanitary landfills will undergo extensive settlements, both total and distortional, which are extremely difficult to predict. Settlements result not only from compression of the underlying materials, but also from the decomposition of organic matter. Gases in landfill areas are health and fire hazards. A thorough study is necessary when utilizing sanitary landfill areas for foundations.

4.5 PEAT AND ORGANIC SOILS. Settlements in these soils are computed in a similar manner as for fine-grained soils. However, the primary consolidation takes place rapidly and the secondary compression continues for a long period of time and contributes much more to the total settlement.

5. TOLERABLE AND DIFFERENTIAL SETTLEMENT

5.1 APPLICATIONS. For an important structure, compute total settlement at a sufficient number of points to establish the overall settlement pattern. From this pattern, determine the maximum scope of the settlement profile or the greatest difference in settlement between adjacent foundation units.

5.2. APPROXIMATE VALUES. Because of natural variation of soil properties and uncertainty on the rigidity of structure and thus actual loads transmitted to foundation units, empirical relationships have been suggested to estimate the differential settlements (or angular distortion) in terms of total settlement (refer to Structure Soil Interaction, by Institution of Civil Engineers). Terzaghi and Peck, page 489) suggested that for footings on sand, differential settlement is unlikely to exceed 75% of the total settlement. For clays, differential settlement may in some cases approach the total settlement.

5.3 TOLERABLE SETTLEMENT.

5.3.1 CRITERIA. Differential settlements and associated rotations and tilt may cause structural damage and could impair the serviceability and function of a given structure. Under certain conditions, differential settlements could undermine the stability of the structure and cause structural failure. Table 4 (Allowable Settlements of Structures, by Bjerrum) provides some guidelines to evaluate the effect of settlement on most structures. Table 5 provides guidelines for tanks and other facilities.

5.3.2 REDUCTION OF DIFFERENTIAL SETTLEMENT EFFECTS. Settlement that can be completed during the early stages of construction, before placing sensitive finishes, generally will not contribute to structural distress. In buildings with light frames where large differential settlements may not harm the frame, make special provisions to avoid damage to utilities or operating equipment. Isolate sensitive equipment, such as motor-generator sets within the structure, on separate rigidly supported foundations. Provide flexible couplings for utility lines at critical locations.

TABLE 4
Tolerable Settlements for Building

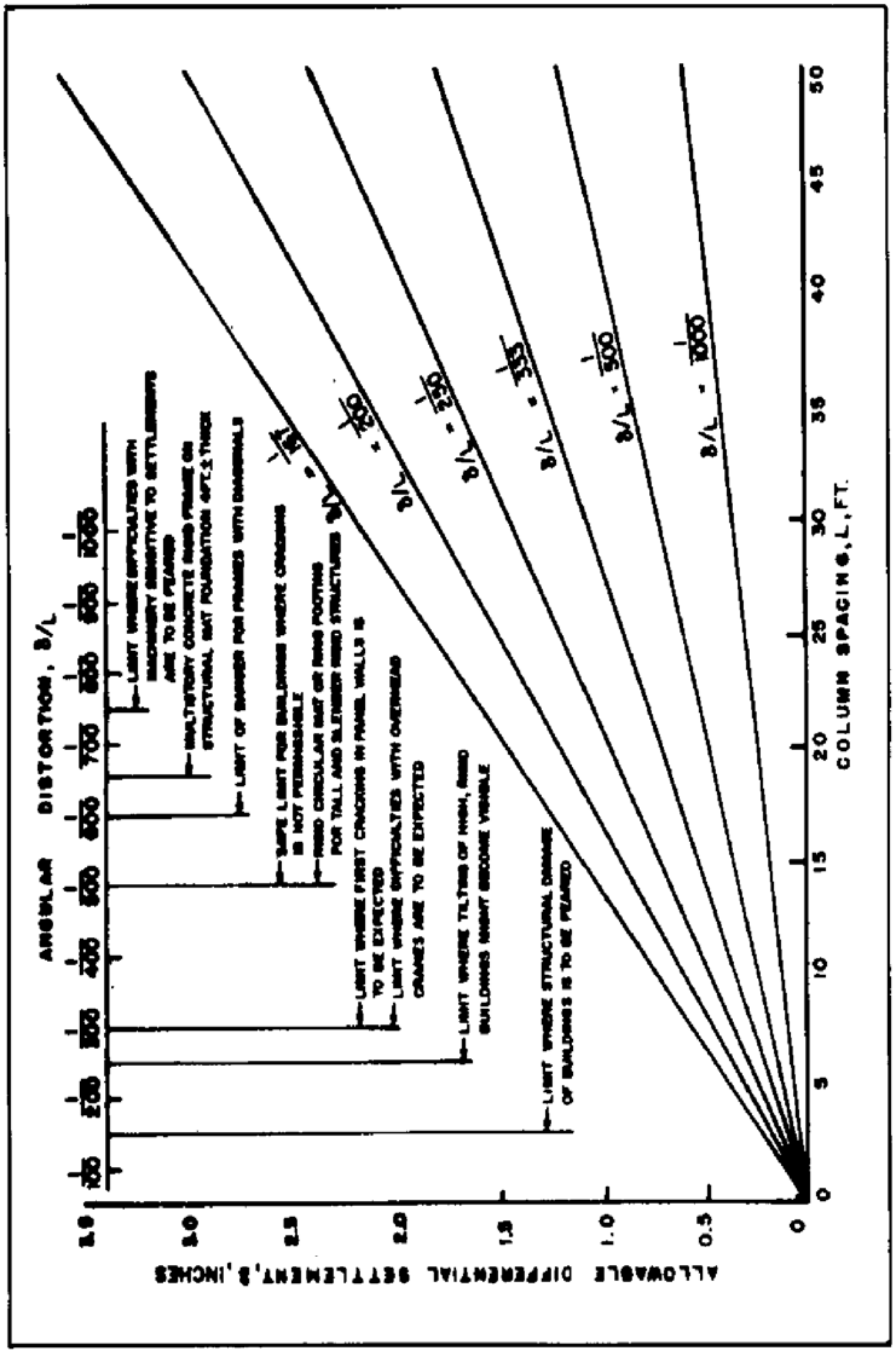
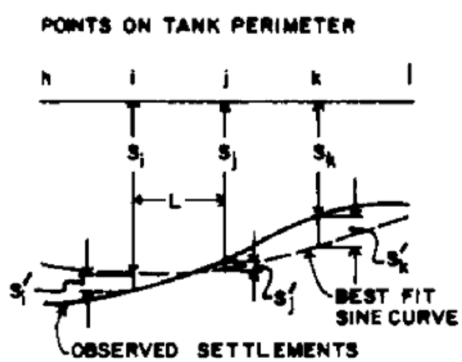


Table 4

Tolerable Settlements for Building

TABLE 5
Tolerable Differential Settlement for Miscellaneous Structures

STRUCTURE		TOLERABLE DISTORTION $\frac{\Delta_{max}}{L}$ or β
A. UNREINFORCED LOAD BEARING WALLS (L and H are respectively length and height of the wall from top of footing)	SAGGING FOR L/H < 3	$\frac{\Delta_{max}}{L}$ = 1/3500 to 1/2500
	FOR L/H > 5	$\frac{\Delta_{max}}{L}$ = 1/2000 to 1/1250
	HOGGING FOR L/H = 1	$\frac{\Delta_{max}}{L}$ = 1/5000
	FOR L/H = 5	$\frac{\Delta_{max}}{L}$ = 1/2500
B. JOINTED RIGID CONCRETE PRESSURE CONDUITS (MAXIMUM ANGLE CHANGE AT JOINT 2 TO 4 TIMES AVERAGE SLOPE OF SETTLEMENT PROFILE. LONGITUDINAL EXTENSION AFFECTS DAMAGE.)		1/65
C. CIRCULAR STEEL PETROLEUM OR FLUID STORAGE TANKS.		β < 1/300 β' = 1/500 to 1/300



$$\beta = \frac{S_i - S_j}{L}$$

$$\beta' = (S_j' - \frac{S_i' + S_k'}{2})\frac{1}{L}$$

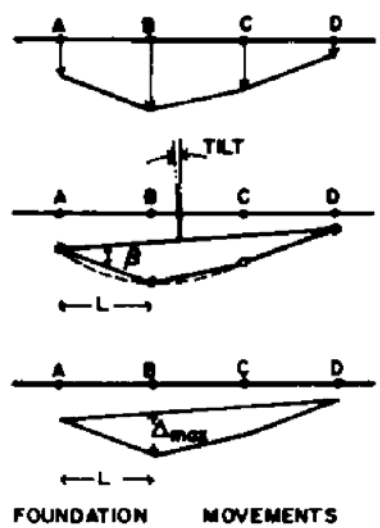


Table 5

Tolerable Differential Settlement for Miscellaneous Structures

5.4 EFFECT OF STRUCTURE RIGIDITY. Computed differential settlement is less accurate than computed total or average settlement because the interaction between the foundation elements and the supporting soil is difficult to predict. Complete rigidity implies uniform settlement and thus no differential settlement. Complete flexibility implies uniform contact pressure between the mat and the soil. Actual conditions are always in between the two extreme conditions. However, depending on the magnitude of relative stiffness as defined below, mats can be defined as rigid or flexible for practical purposes.

5.4.1 UNIFORMLY LOADED CIRCULAR RAFT. In the case where the raft has a frictionless contact with an elastic half space (as soil is generally assumed to represent), the relative stiffness is defined as:

R = radius of the raft, t = thickness of raft, subscripts r and s refer to raft and soil, υ = Poission's ratio and E = Young's modulus.

For $K_r \le 0.08$, raft is considered flexible and for $K_r \le 5.0$ raft s considered rigid. For intermediate stiffness values see Numerical Analyses of Uniformly Loaded Circular Rafts on Elastic Layers of Finite Depth, by Brown.

5.4.2 UNIFORMLY LOADED RECTANGULAR RAFT. For frictionless contact between the raft and soil, the stiffness factor is defined as:

B = width of the foundation. Other symbols are defined in 5.4.1.

For $K_r \le 0.05$, raft is considered flexible and for K+r, ≥ 10, raft is considered rigid. For intermediate stiffness values see Numerical Analysis of Rectangular Raft on Layered Foundations, by Frazer and Wardle.

6. METHODS OF REDUCING OR ACCELERATING SETTLEMENT

6.1 GENERAL. See Table 6 for methods of minimizing consolidation settlements. These include removal or displacement of compressible material and preconsolidation in advance of final construction.

6.2 REMOVAL OF COMPRESSIBLE SOILS. Consider excavation or displacement of compressible materials for stabilization of fills that must be placed over soft strata.

Method	Comment
Procedures for linear fills on swamps or compressible surface stratum:	
Excavation of soft material	When compressible foundation soils extend to depth of about 10 to 15 ft, it may be practicable to remove entirely. Partial removal is combined with various methods of displacing remaining soft material.
Displacement by weight of fill	Complete displacement is obtained only when compressible foundation is thin and very soft. Weight displacement is combined with excavation of shallow material.
Jetting to facilitate displacement	For a sand or gravel fill, jetting within the fill reduces its rigidity and promotes shear failure to displace soft foundation. Jetting within soft foundation weakens it to assist in displacement.
Blasting by trench or shooting methods	Charge is placed directly in front of advancing fill to blast out a trench into which the fill is forced by the weight of surcharge built up at its point. Limited to depths not exceeding about 20 ft.
Blasting by relief method	Used for building up fill on an old roadway or for fills of plastic soil. Trenches are blasted at both toes of the fill slopes, relieving confining pressure and allowing fill to settle and displace underlying soft materials
Blasting by underfill method	Charge is placed in soft soil underlying fill by jetting through the fill at a preliminary stage of its buildup. Blasting loosens compressible material, accelerating settlement and facilitating displacement to the sides. In some cases relief ditches are cut or blasted at toe of the fill slopes. Procedure is used in swamp deposits up to 30 ft thick.

Table 6

Methods of Reducing or Accelerating Settlement or Coping with Settlement

Method	Comment
Procedures for preconsolidation of soft foundations	
Surcharge fill	Used where compressible stratum is relatively thin and sufficient time is available for consolidation under surcharge load. Surcharge material may be placed as a stockpile for use later in permanent construction. Soft foundation must be stable against shear failure under surcharge load.
Accelerating consolidation by vertical drains	Used where tolerable settlement of the completed structure is small, where time available for preconsolidation is limited, and surcharge fill is reasonably economical. Soft foundation must be stable against shear failure under surcharge load.
Vertical sand drains with or without surcharge fill	Used to accelerate the time for consolidation by providing shorter drainage paths.
Wellpoints placed in vertical sand drains	Used to accelerate consolidation by reducing the water head, thereby permitting increased flow into the sand drains. Particularly useful where potential instability of soft foundation restricts placing of surcharge or where surcharge is not economical.
Vacuum method	Variation of wellpoint in vertical sand drain but with a positive seal at the top of the sand drain surrounding the wellpoint pipe. Atmospheric pressure replaces surcharge in consolidating soft foundations.
Balancing load of structure by excavation	Utilized in connection with mat or raft foundations on compressible material or where separate spread footings are founded in suitable bearing material overlying compressible stratum. Use of this method may eliminate deep foundations, but it requires very thorough analysis of soil compressibility and heave.

Table 6 (continued)

Methods of Reducing or Accelerating Settlement or Coping with Settlement

6.2.1 REMOVAL BY EXCAVATION. Organic swamp deposits with low shear strength and high compressibility should be removed by excavation and replaced by controlled fill. Frequently these organic soils are underlain by very loose fine sands or silt or soft clayey silts which may be adequate for the embankment foundation and not require replacement. Topsoil is usually stripped prior to placement of fills; however, stripping may not be required for embankments higher than 6 feet as the settlement from the upper 1/2 foot of topsoil is generally small and takes place rapidly during construction period. However, if the topsoil is left in place, the overall stability of the embankment should be checked assuming a failure plane through the topsoil.

6.2.2 DISPLACEMENT. Partial excavation may be accompanied by displacement of the soft foundation by the weight of fill. The advancing fill should have a steep front face. The displacement method is usually used for peat and muck deposits. This method has been used successfully in a few cases for soft soils up to 65 feet deep. Jetting in the fill and various blasting methods are used to facilitate displacement. Fibrous organic materials tend to resist displacement resulting in trapped pockets which may cause differential settlement.

6.3 BALANCING LOAD BY EXCAVATION. To decrease final settlement, within an excavation that is carried to a depth at which the weight of overburden, removed partially or completely, balances the applied load.

6.3.1 COMPUTATION OF TOTAL SETTLEMENT. In this case, settlement is derived largely from recompression. The amount of recompression is influenced by magnitude of heave and magnitude of swell in the unloading stage.

6.3.2 EFFECT OF DEWATERING. If drawdown for dewatering extends well below the planned subgrade, heave and consequent recompression are decreased by the application of capillary stresses. If groundwater level is restored after construction, the load removed equals the depth of excavation times total unit weight of the soil. If groundwater pressures are to be permanently relieved, the load removed equals the total weight of soil above the original water table plus the submerged weight of soil below the original water table. Calculate effective stresses as described in Figure 2, and consolidation under structural loads as shown in Figure 3.

6.4 PRECONSOLIDATION BY SURCHARGE. This procedure causes a portion of the total settlement to occur before construction. It is used primarily for fill beneath paved areas or structures with comparatively light column loads. For heavier structures, a compacted fill of high rigidity may be required to reduce stresses in compressible foundation soil.

6.4.1 ELIMINATION OF PRIMARY CONSOLIDATION. Use Figure 17 to determine surcharge load and percent consolidation under surcharge necessary to eliminate primary consolidation under final load. This computation assumes that the rate of consolidation under the surcharge is equal to that under final load.

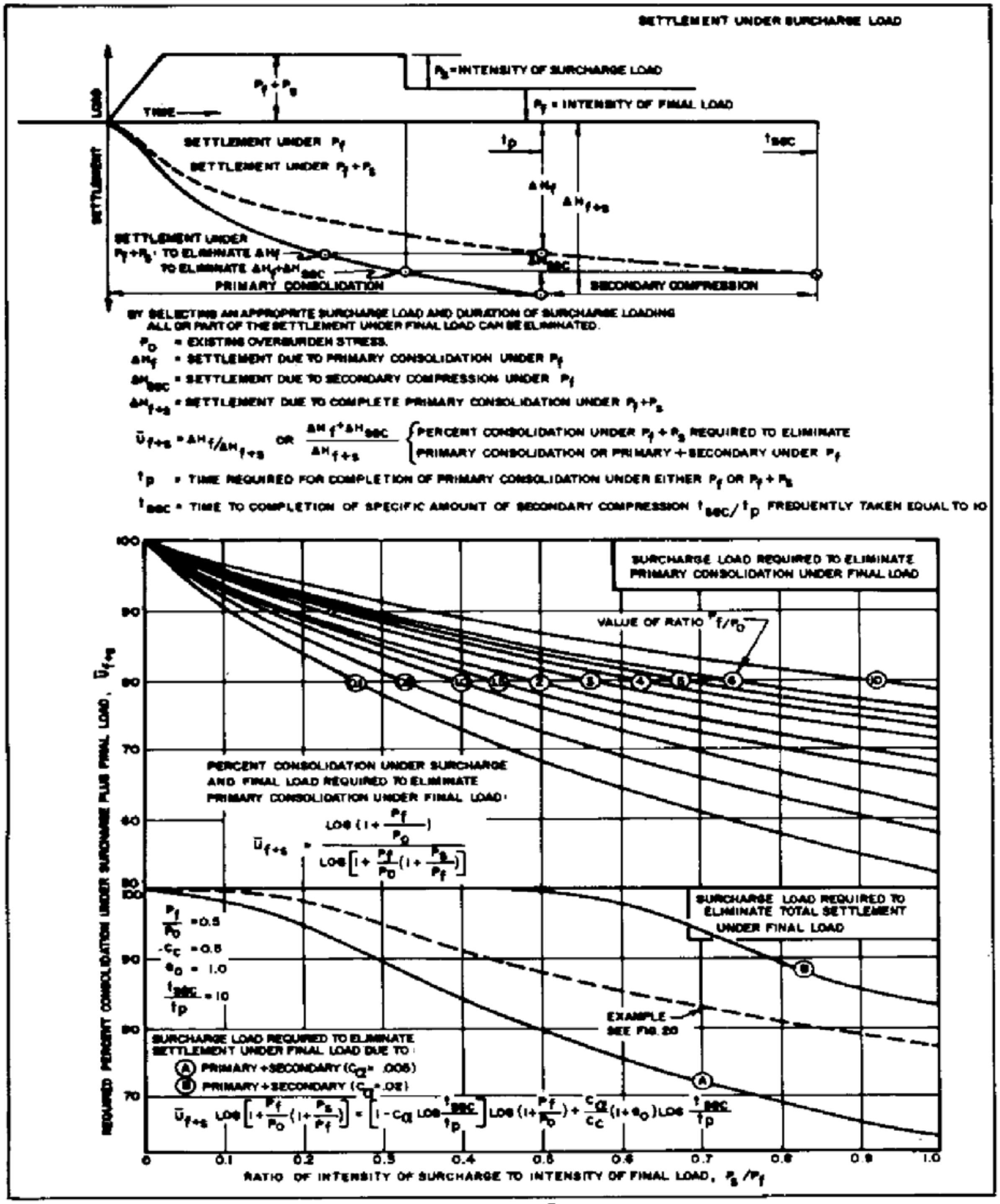


FIGURE 17

Surcharge Load Required to Eliminate Settlement Under Final Load

Figure 17

Surcharge Load Required to Eliminate Settlement Under Load

6.4.2 ELIMINATION OF SECONDARY CONSOLIDATION. Use the formula in the bottom panel of Figure 17 to determine surcharge load and percent consolidation under surcharge required to eliminate primary consolidation plus a specific secondary compression under final load.

6.4.3 LIMITATIONS ON SURCHARGE. In addition to considerations of time available and cost, the surcharge load may induce shear failure of the soft foundation soil. Analyze stability under surcharge.

6.5 VERTICAL DRAINS. These consist of a column of pervious material placed in cylindrical vertical holes in the compressible stratum at sufficiently close spaces so that the horizontal drainage path is less than the vertical drainage path. All drains should be connected at the ground surface to a drainage blanket. Vertical drains are utilized in connection with fills supporting pavements or low- to moderate-load structures and storage tanks. Common types of vertical drains are shown in Table 7 (refer to Use of Precompression and Vertical Sand Drains for Stabilization of Foundation Soils, by Ladd). Sand drains driven with a closed-end pipe produce the largest displacement and disturbance in the surrounding soil and thus their effectiveness is reduced.

6.5.1 CHARACTERISTICS. Vertical drains accelerate consolidation by facilitating drainage of pore water but do not change total compression of the stratum subjected to a specific load. Vertical drains are laid out in rows, staggered, or aligned to form patterns of equilateral triangles or squares. See Figure 18 for cross-section and design data for typical installation for sand drains.

6.5.2 CONSOLIDATION RATE. Time rate of consolidation by radial drainage of pore water to vertical drains is defined by time factor curves in upper panel of Figure 10. For convenience, use the nomograph of Figure 19 to determine consolidation time rate. Determine the combined effect of vertical and radial drainage on consolidation time rate as shown in the example in Figure 10.

6.5.3 VERTICAL DRAIN DESIGN. See Figure 20 for an example of design. For a trial selection of drain diameter and spacing, combine percent consolidation at a specific time from vertical drainage with percent consolidation for radial drainage to the drain. This combined percent consolidation U_C is plotted versus elapsed time for different drain spacing in the center panel of Figure 20. Selection of drain spacing depends on the percent consolidation required prior to start of structure, the time available for consolidation, and economic considerations.

6.5.4 ALLOWANCE FOR SMEAR AND DISTURBANCE. In cases where sand drain holes are driven with a closed-end pipe, soil in a surrounding annular space one-third to one-half the drain diameter in width is remolded and its stratification is distorted by smear. Smear tends to reduce the horizontal permeability coefficient, and a correction should be made in accordance with Figure 21.

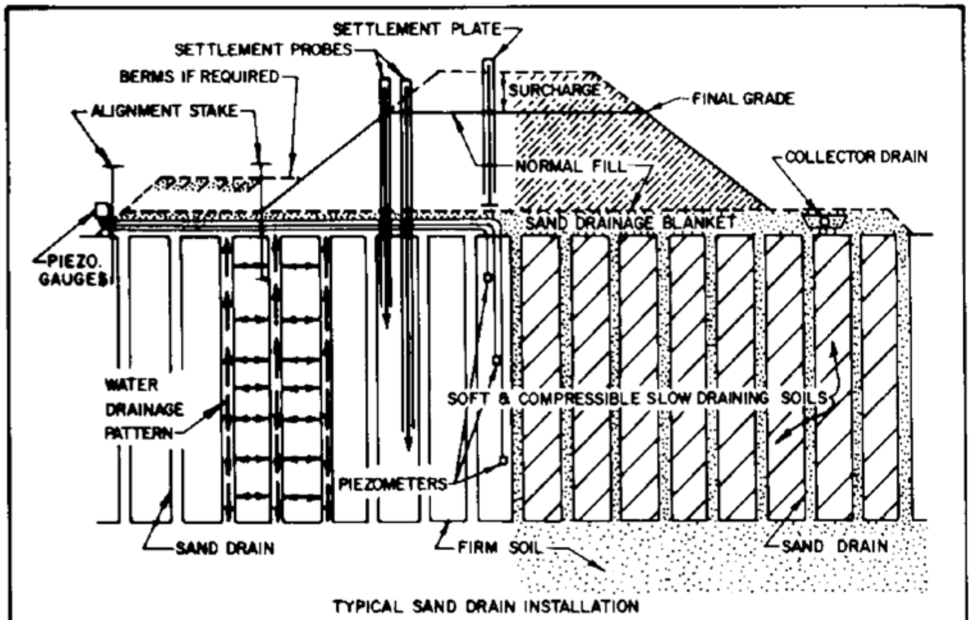
TABLE 7
Common Types of Vertical Drains

General Type	Sub-type	Typical Installation	
		d_w	s
1. Driven Sand Drain	Closed end mandrel	$18^{\pm}$ in	5 - 20 ft
2. Augered Sand Drain	(a) Screw type auger	6 - 30 in	-
	(b) Continuous flight hollow stem auger	18 in	5 - 20 ft
3. Jetted Sand Drain	(a) Internal jetting	18 in	5 - 20 ft
	(b) Rotary jet	12 - 18 in	5 - 20 ft
	(c) Dutch jet-bailer	12 in	4 - 16 ft
4. "Paper" Drain	(a) Kjellman cardboard wick	$0.1^{\pm}$ in by $4^{\pm}$ in	1.5^{+} - 4^{+} ft
	(b) Cardboard coated plastic wick	slightly thicker	-
5. Fabric Encased Sand Drain	(a) Sandwick	2.5 - 3 in	4 - 12 ft
	(b) Fabridrain	5 in	-

d_w = diameter of drain, s = drain spacing

Table 7

Common Types of Vertical Drains



DIAMETER OF DRAINS RANGES FROM 6 TO 30 IN., GENERALLY BETWEEN 16 AND 20 IN.

SPACING OF DRAINS RANGES FROM 6 TO 20 FT ON CENTER, GENERALLY BETWEEN 6 AND 10 FT.

PRINCIPAL METHODS FOR INSTALLING DRAINS ARE CLOSED OR OPEN MANDRELS ADVANCED BY DRIVING OR JETTING, OR ROTARY DRILLING WITH OR WITHOUT JETS. DRIVING CLOSED MANDREL IS THE MOST COMMON.

DRAIN BACKFILL MATERIAL SHOULD HAVE SUFFICIENT PERMEABILITY TO DISCHARGE PORE WATER FLOW ANTICIPATED, BUT USUALLY DOES NOT MEET FILTER REQUIREMENTS AGAINST FOUNDATIONS SOILS. CLEAN SANDS WITH NO MORE THAN 3% BY WEIGHT PASSING No. 200 SIEVE IS USUALLY SUITABLE. A TYPICAL GRADATION IS AS FOLLOWS:

SIEVE No.	4	16	50	100	200
% FINER BY WEIGHT:	90-100	40-85	2-30	0-7	0-3

SAND DRAINAGE BLANKET MATERIAL IS SIMILAR TO THAT USED FOR DRAIN BACKFILL. IN SOME CASES GRAVEL WINDROWS OR PERFORATED, CORRUGATED, METAL PIPE ARE PLACED IN DRAINAGE BLANKET TO REDUCE HEAD LOSS IN DRAINAGE BLANKET. LONGITUDINAL DITCH OR COLLECTOR DRAIN MAY BE PLACED AT TOE. GRANULAR WORKING MAT IS SOMETIMES PLACED BELOW DRAINAGE BLANKET TO SUPPORT EQUIPMENT.

SURCHARGE LOAD IS PLACED TO REDUCE OR ELIMINATE POSTCONSTRUCTION CONSOLIDATION BENEATH NORMAL FILL. GENERALLY THE SURCHARGE LOAD IS NO MORE THAN ABOUT 30% OF NORMAL EMBANKMENT LOAD.

FIELD CONTROL DEVICES:

PIEZOMETERS OF STANDPIPE OR CLOSED SYSTEM TYPE TO OBSERVE PORE WATER PRESSURES;

SETTLEMENT PLATES, MINIMUM 3 FT SQUARE, PLACED AT BASE OF FILL TO RECORD TOTAL SETTLEMENT;

SETTLEMENT PROBES DRIVEN OR AUGERED INTO FOUNDATION STRATUM TO MEASURE COMPRESSION WITHIN FOUNDATION.

ALIGNMENT STAKES, T-SHAPED STAKES PLACED AT OR OUTSIDE EMBANKMENT TOE TO OBSERVE LATERAL MOVEMENT AND HEAVE.

FIGURE 18
Data for Typical Sand Drain Installation

Figure 18

Data for Typical Sand Drain Installation

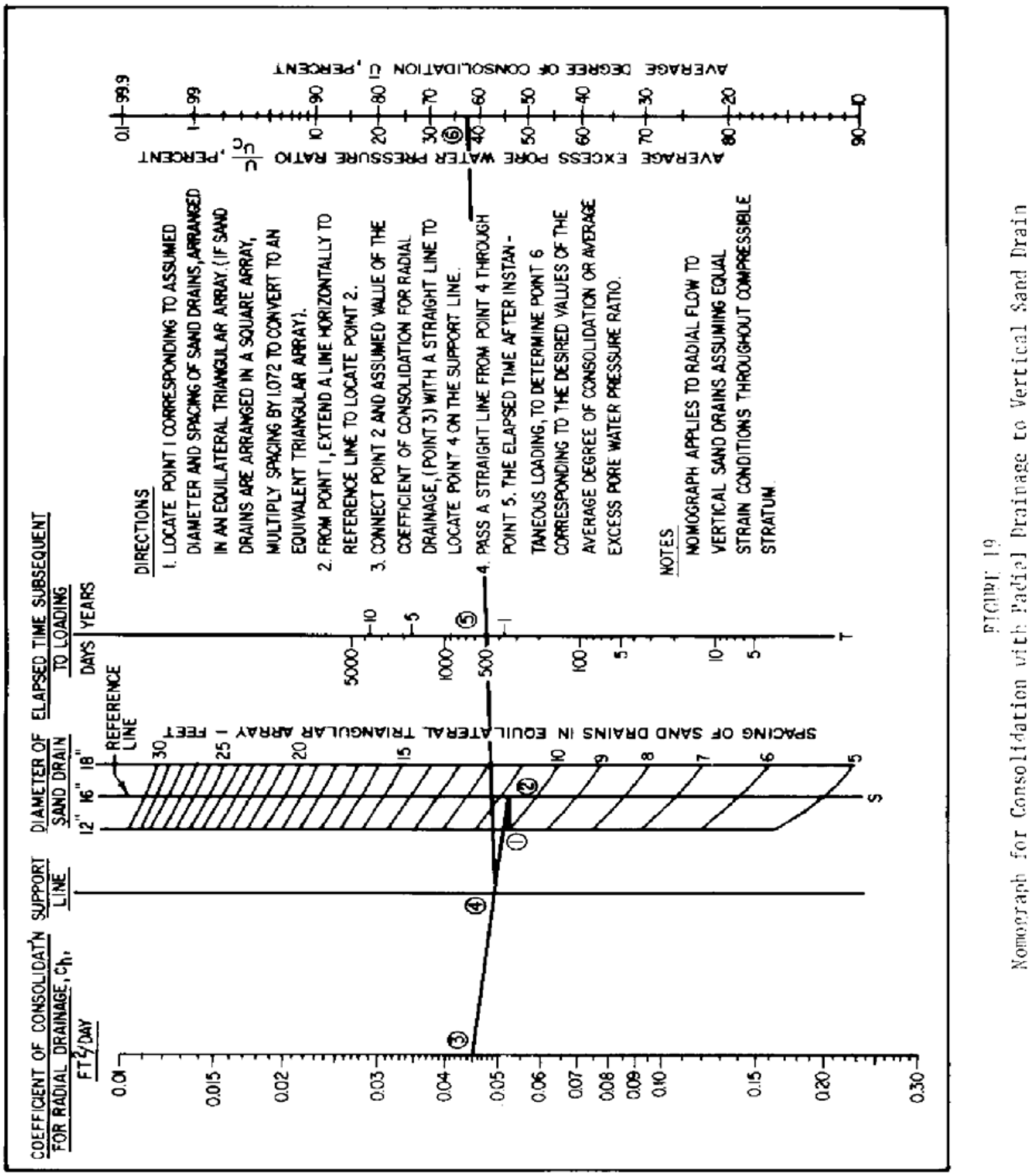


Figure 19

Nomograph for Consolidation with Radial Drainage to Vertical and Drain

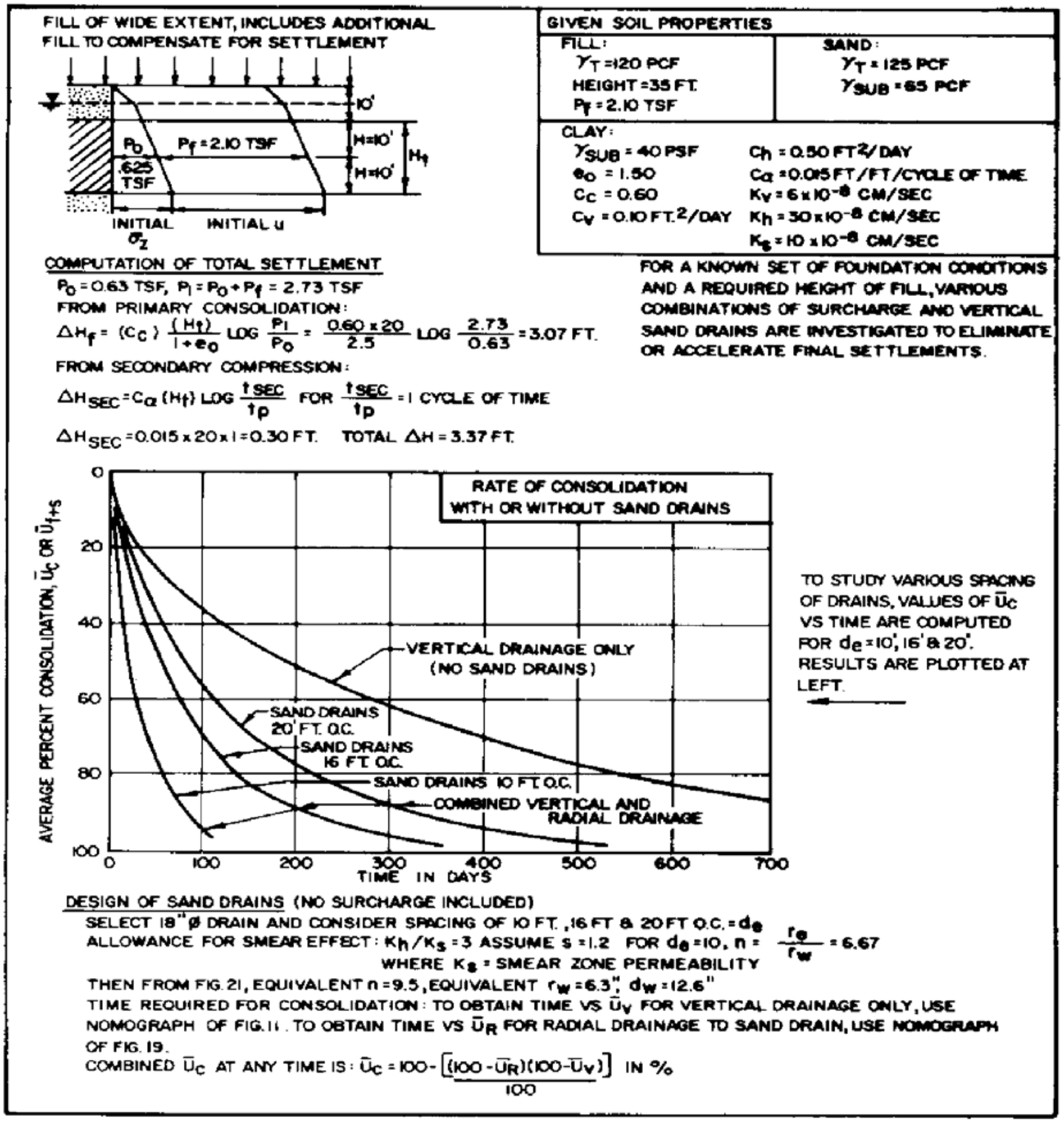
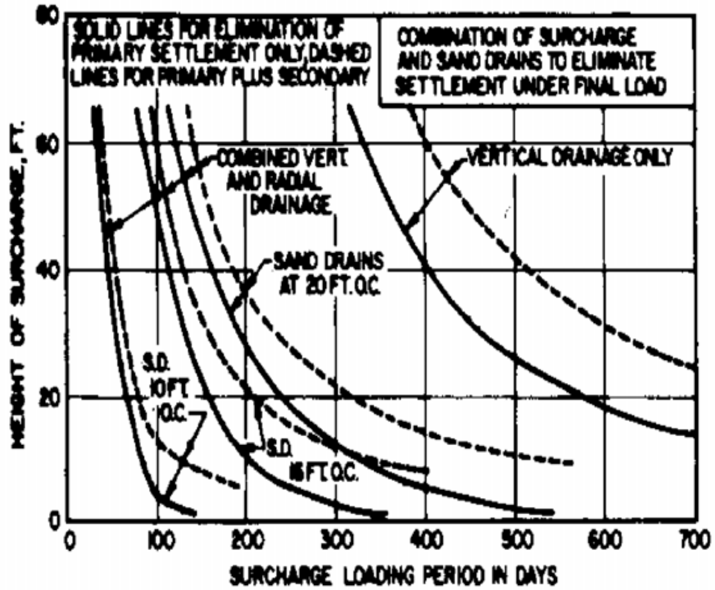


FIGURE 20
Example of Surcharge and Sand Drain Design

Figure 20

Example of Surcharge and Sand Drain Design



SELECTION OF SURCHARGE HEIGHT

$\Delta H_f = 3.07'$, $\Delta H_{SEC.} = 0.30'$, $P_f/P_0 = 3.36$

TO ELIMINATE SETTLEMENT UNDER P_f, $\bar{U}_C$ IS TAKEN EQUAL TO $\bar{U}_{f+s}$, $\bar{U}_{f+s} = \dfrac{\Delta H_f}{\Delta H_{f+s}}$ OR $\dfrac{\Delta H_f + \Delta H_{SEC}}{\Delta H_{f+s}}$

RELATION OF $\bar{U}_{f+s}$ AND TIME IS GIVEN ABOVE FOR VARIOUS DRAIN SPACINGS.

SURCHARGE P_s FOR VALUES OF $\bar{U}_{f+s} = \dfrac{\Delta H_f}{\Delta H_{f+s}}$ IS GIVEN IN FIG. 17

SURCHARGE P_s FOR VALUES OF $\bar{U}_{f+s} = \dfrac{\Delta H_f + \Delta H_{SEC}}{\Delta H_{f+s}}$ IS GIVEN BY FORMULA IN FIG. 17

USING THESE RELATIONSHIPS, P_s (EXPRESSED AS HEIGHT OF SURCHARGE) REPLACES $\bar{U}_{f+s}$ IN FIGURE 17.

COMBINATION OF SAND DRAIN AND SURCHARGE IS SELECTED BASED ON TIME AVAILABLE AND COMPARATIVE COSTS.

FIGURE 20 (continued)
Example of Surcharge and Sand Drain Design

Figure 20 (continued)
Example of Surcharge and Sand Drain Design

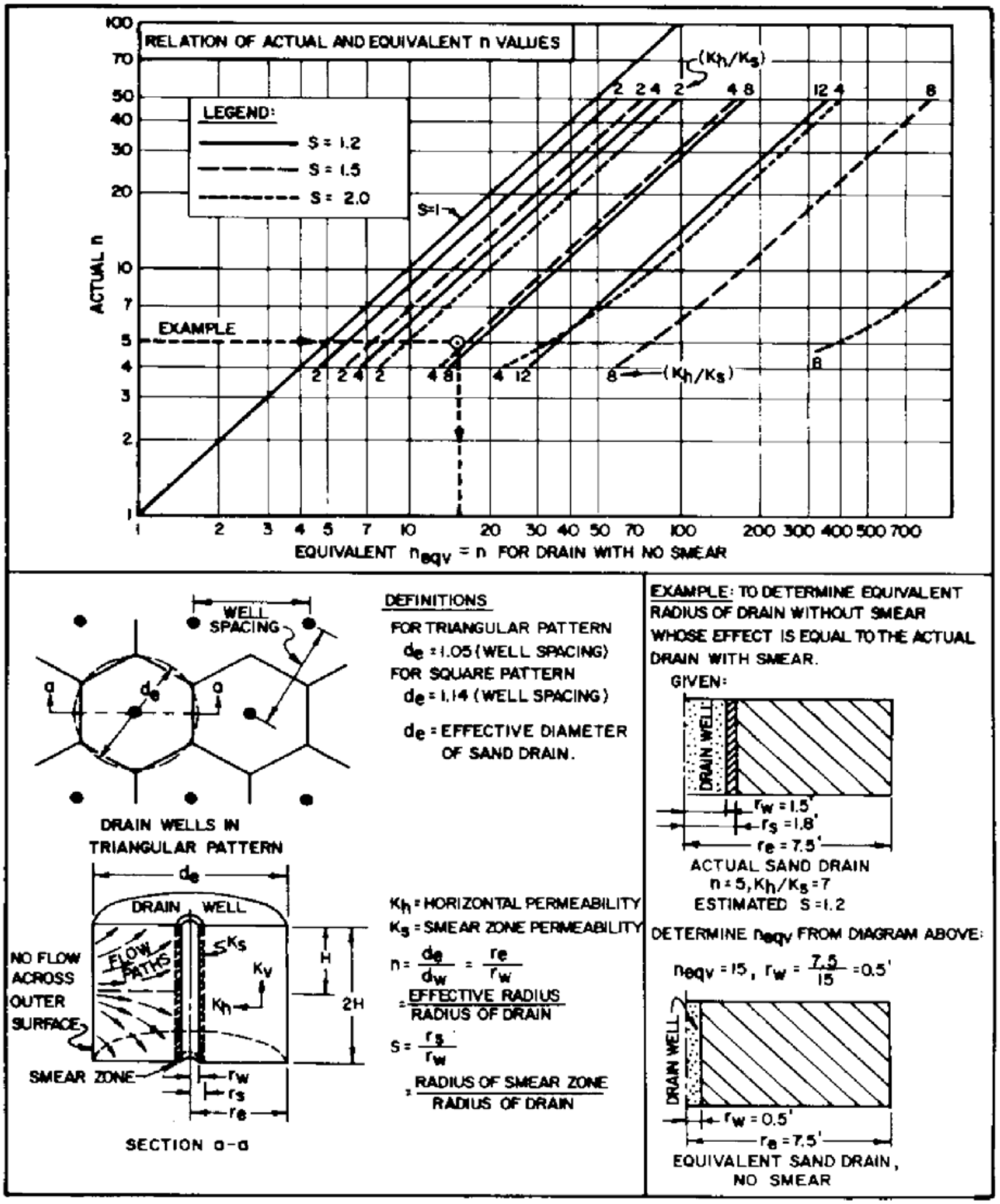


FIGURE 21
Allowance for Smear Effect in Sand Drain Design

Figure 21

Allowance for Smear Effect in Sand Drain Design

6.5.5 SAND DRAINS PLUS SURCHARGE. A surcharge load is normally placed above the final fill level to accelerate the required settlement. Surcharge is especially necessary when the compressible foundation contains material in which secondary compression predominates over primary consolidation. The percent consolidation under the surcharge fill necessary to eliminate a specific amount of settlement under final load is determined as shown in the lowest panel of Figure 20.

6.5.6 GENERAL DESIGN REQUIREMENTS. Analyze stability against foundation failure, including the effect of pore pressures on the failure plane. Determine allowable buildup of pore pressure in the compressible stratum as height of fill is increased.

6.5.6.1 HORIZONTAL DRAINAGE. For major installation investigate in detail the horizontal coefficient of consolidation by laboratory tests with drainage in the horizontal direction, or field permeability tests to determine horizontal permeability.

6.5.6.2 CONSOLIDATION TESTS. Evaluate the importance of smear or disturbance by consolidation tests on remolded samples. For sensitive soils and highly stratified soils, consider nondisplacement methods for forming drain holes.

6.5.6.3 DRAINAGE MATERIAL. Determine drainage material and arrangement to handle maximum flow of water squeezed from the compressible stratum

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6.5.7 CONSTRUCTION CONTROL REQUIREMENTS. Control the rate of fill rise by installing piezometer and observing pore pressure increase for comparison with pore pressure values compatible with stability. Check anticipated rate of consolidation by pore pressure dissipation and settlement measurements.

7. ANALYSIS OF VOLUME EXPANSION.

7.1 CAUSES OF VOLUME EXPANSION. Volume expansion is caused by (a) reduction of effective stresses, (b) mineral changes, and (c) formation and growth of ice lenses. Swell with decrease of effective stress is a reverse of the consolidation process. For description of swelling problems and suggested treatment, see Table 8. Where highly preconsolidated plastic clays are present at the ground surface, seasonal cycles of rainfall and desiccation produce volume changes. The most severe swelling occurs with montmorillinite clays although, in an appropriate climate, any surface clay of medium to high plasticity with relatively low moisture content can heave.

7.2 MAGNITUDE OF VOLUME EXPANSION. Figure 22 outlines a procedure for estimating the magnitude of swelling that may occur when footings are built on expansive clay soils. This figure also indicates a method of determining the necessary undercut to reduce the heave to an acceptable value.

TABLE 8
Heave From Volume Change

Conditions and materials	Mechanism of heave	Treatment
Reduction of effective stress of overburden:		
Temporary reduction of effective stress by excavation for structure foundation in preconsolidated clays.	Soil swells in accordance with laboratory e-p curves. Heave is maximum at center of excavation. Total potential heave may not have occurred by time the load is reapplied. Final structural load will recompress foundation materials.	Provide drainage for rapid collection of surface water. Avoid disturbance to subgrade by placing 4-in.-thick working mat of lean concrete immediately after exposing subgrade. Heave is minimized if the groundwater is drawn down 3 or 4 ft below base of excavation at its center to maintain capillary stresses.
Permanent reduction of effective stress by excavation in chemically inert, uncemented clay-shale or shale.	In sound shale where water cannot obtain access to the shale, swelling may be insignificant.	Protect shale from wetting and drying during excavation by limiting area opened at subgrade and with concrete working mat. Pour concrete floors and foundations directly on protected shale with no underfloor drainage course. Backfill around walls with impervious soils to prevent access of water. Provide proper surface drainage and paving if necessary to avoid infiltration.
	For hydraulic structures or construction below the ground water table, reduction of effective stresses will cause permanent heave in accordance with laboratory e-p curves. Alternate wetting and drying during excavation increases swelling potential.	Where an increase in water content is probable, special structural designs must be considered. These include (1) anchoring or rock bolting the floor to a depth in shale that provides suitable hold down against swelling pressures; (2) a floor supported on heavily loaded column footings with an opening or compressible filler beneath floors; and (3) a mat foundation designed to resist potential swelling pressures. In any case, excavation in the shale should be protected by sealing coats or working mat immediately after exposure at subgrade.
Reduction of effective stress of overburden and release of capillary stress:		
Construction of earth dams of heavily compacted plastic clays.	Intrusion of seepage from reservoir releases capillary pressures and reduces effective stress of overburden and may produce swelling leading to sloughing of the slopes. Most critical material are CH clays with swelling index exceeding 0.07. Compaction at relatively low water contents, where the water deficiency in the clay mineral lattice is high and the degree of saturation is low, will accentuate swelling.	Avoid placing highly plastic fill on or near embankment slopes. Compact clays at a relatively high moisture content consistent with strength and compressibility requirements. Avoid overcompaction to an unnecessarily high dry unit weight.
Construction of structural fill for light buildings of compacted plastic clay.	Rise of groundwater, seepage, leakage, or elimination of surface evaporation increases degree of saturation and reduces effective stress, leading to expansion.	Compact clays as wet as practicable consistent with compressibility requirements. Avoid overcompaction of general fill and undercompaction of backfill at column footings or in utility trenches which would accentuate differential movements. Stabilization of compacted fills with various salt admixtures reduces swelling potential by increasing ion concentration in pore water.

Table 8

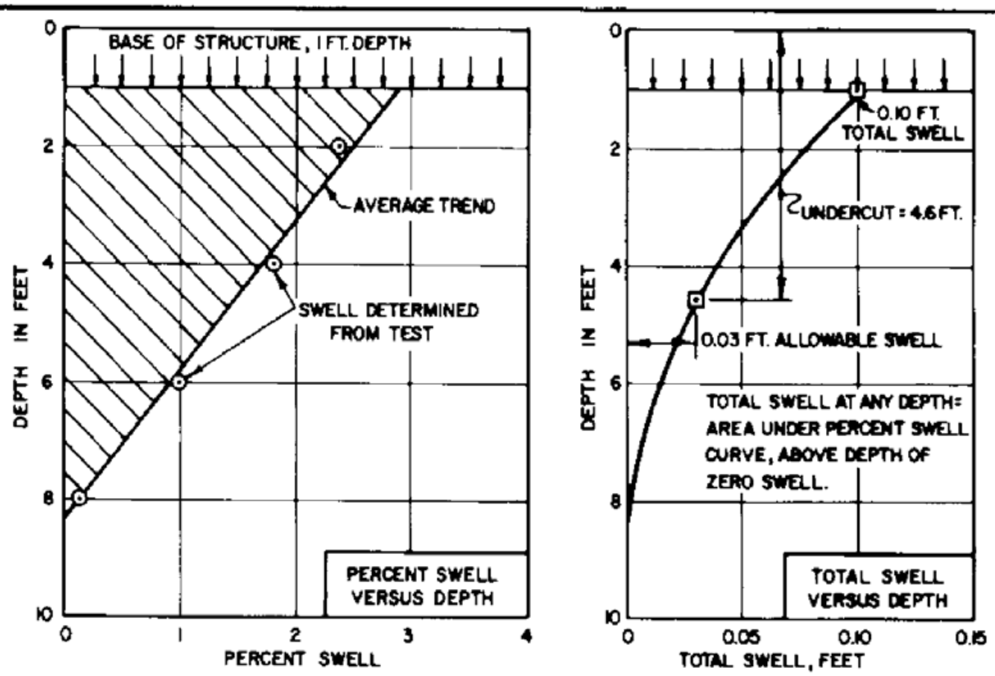
Heave from Volume Change

TABLE 8 (continued)
Heave From Volume Change

Conditions and materials	Mechanism of heave	Treatment
Changes of capillary stresses:		
Construction of light buildings on surface strata of highly preconsolidated clays in temperate climates subject to substantial seasonal fluctuations in rainfall. (Southern England, as an example.)	Seasonal movements 1 or 2 in. upwards and downwards occur within the upper 3 to 5 ft. Settlement occurs in early summer and expansion in the fall. Caused by change of capillary stresses produced by transpiration to nearby trees, plant, or grass cover surrounding the structure. Movements are maximum at edge of building. Groundwater is shallow. Change of capillary stresses by evaporation is not of prime importance.	Light reinforcing or stiffening minimize effects in small houses. Basements carried to usual depths usually eliminate movements.
Construction of light buildings on clays of high activity, highly preconsolidated with fractures and slickensides, in climate where hot summers alternate with wet winters. (South-central Texas for example.)	Even in the absence of vegetal cover, seasonal cycles of settlement and heave occur because of the alternate increase and release of capillary stresses. Buildings constructed during wet season may undergo small but nonuniform settlement beneath exterior footings. Buildings constructed in the dry season undergo uneven heave up to 3 or 4 in. maximum, distributed irregularly over the structure.	Support light footings and slabs on compacted, coarse-grained fill about 4 to 6 ft thick. Pave peripheral areas to minimize subsoil moisture content change. Consider the use of belled caissons with supported floor. Open block wall foundations have been utilized for small houses. Collect rainwater falling on structure and surrounding areas and convey runoff away from structures.
Construction of light to medium load structures in hot, arid climate where the free surface evaporation is several times larger than annual rainfall. Difficulties are greatest in fractured and slickensided clay of high activity, with low water table and maximum deficiency of evaporation over rainfall. (South Africa, as an example.)	Permanent moisture deficiency exists in the ground. Construction eliminates evaporation over building area, reducing capillary stresses and causing movement of moisture to beneath building. This leads to continuing heave with minor seasonal fluctuations. Thermoosmotic gradients directed toward cooled subsoil beneath structure contribute to increase in moisture, which may extend to depths of 10 to 15 ft.	Damage is minimized by use of slab or raft foundation, dry wall construction, steel or reinforced concrete framing, reinforced foundation beams, and provision for jacking. Heave is eliminated by removal of desiccated material to a depth of 8 to 12 ft.and replacement by granular fill; or belled caissons, founded near the water table and reinforced to resist tensile forces, supporting floor between caissons with opening or compressible filler beneath floors. Divert rainwater and surface runoff away from structure.
Chemical changes:		
Excavation and exposure of clay-shales or shales containing pyrite (iron sulphide) or anhydrite (calcium sulphate).	Exposure to air and water causes oxidation and hydration of pyrites with a volumetric expansion of as much as ten times their original volume, or hydration of anhydrite to gypsum.	Rough excavate no closer than one-half foot to final subgrade and protect exposed shale with a spray or mop coat of bitumen. When ready for foundations, excavate to final grade and pour concrete immediately over a spray or mop coat of bitumen.

Table 8 (continued)

Heave from Volume Change



MATERIALS INVESTIGATED ARE CLAYS, HIGHLY OVERCONSOLIDATED BY CAPILLARY STRESSES THAT ARE EFFECTIVE PRIOR TO THE CONSTRUCTION OF THE STRUCTURE UPON THEM.

PROCEDURE FOR ESTIMATING TOTAL SWELL UNDER STRUCTURE LOAD.

1. OBTAIN REPRESENTATIVE UNDISTURBED SAMPLES OF THE SHALLOW CLAY STRATUM AT A TIME WHEN CAPILLARY STRESSES ARE EFFECTIVE; I.E., WHEN NOT FLOODED OR SUBJECTED TO HEAVY RAIN.
2. LOAD SPECIMENS (AT NATURAL WATER CONTENT) IN CONSOLIDOMETER UNDER A PRESSURE EQUAL TO THE ULTIMATE VALUE OF OVERBURDEN FOR HIGH GROUND WATER, PLUS WEIGHT OF STRUCTURE. ADD WATER TO SATURATE AND MEASURE SWELL.
3. COMPUTE FINAL SWELL IN TERMS OF PERCENT OF ORIGINAL SAMPLE HEIGHT AND PLOT SWELL VERSUS DEPTH, AS IN THE LEFT PANEL.
4. COMPUTE TOTAL SWELL WHICH IS EQUAL TO THE AREA UNDER THE PERCENT SWELL VERSUS DEPTH CURVE. FOR THE ABOVE EXAMPLE:

   TOTAL SWELL = 1/2 (8.2 − 1.0) x 2.8/100 = 0.10 FT.

PROCEDURE FOR ESTIMATING UNDERCUT NECESSARY TO REDUCE SWELL TO AN ALLOWABLE VALUE.

1. FROM PERCENT SWELL VERSUS DEPTH CURVE PLOT RELATIONSHIP OF TOTAL SWELL VERSUS DEPTH AT THE RIGHT. TOTAL SWELL AT ANY DEPTH EQUALS AREA UNDER THE CURVE AT LEFT, INTEGRATED UPWARD FROM THE DEPTH OF ZERO SWELL.
2. FOR A GIVEN ALLOWABLE VALUE OF SWELL, READ THE AMOUNT OF UNDERCUT NECESSARY FROM THE TOTAL SWELL VERSUS DEPTH CURVE. FOR EXAMPLE, FOR AN ALLOWABLE SWELL OF 0.03 FT, UNDERCUT REQUIRED = 4.6 FT. UNDERCUT CLAY IS REPLACED BY AN EQUAL THICKNESS OF NONSWELLING COMPACTED FILL.

FIGURE 22
Computation of Swell of Desiccated Clays

Figure 22

Computation of Swell of Desiccated Clay

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