CHAPTER 4
EVALUATION OF SETTLEMENT FOR DYNAMIC AND TRANSIENT LOADS

4-1. General. Dynamic and transient forces cause particle rearrangements and can cause considerable settlement, particularly in cohesionless soils, when the particles move into more compact positions. A large portion of dynamic live forces applied to foundation soil is from traffic on pavements. Dynamic forces from a rolling wheel depressing a pavement cause a multidirectional combination of cyclic shear and compression strains that precludes presentation of an appropriate settlement analysis in this chapter. This chapter provides guidance for analysis of settlement from earthquakes and repeated loads.

a. Amount of Settlement. The amount of settlement depends on the initial density of the soil, thickness of the soil stratum, and the maximum shear strain developed in the soil. Cohesionless soils with relative densities D_r greater than about 75 percent should not develop significant settlement; however, intense dynamic loading can cause some settlement of 1 to 2 percent of the stratum thickness even in dense sands.

b. Cause of Differential Settlement. A major cause of differential settlement is the compaction of loose sands during dynamic loading. Vibrations caused by machinery often cause differential settlement that may require remedial repairs or limitations on machine operations.

c. Time Effects. Time required for settlement from shaking can vary from immediately to almost a day. Settlement in dry sands occurs immediately during shaking under constant effective vertical stress. Shaking of saturated sands induces excess pore water pressures which lead to settlement when the pore pressures dissipate.

d. Accuracy. Errors associated with settlement predictions from dynamic loads will exceed those for static loads and can be 50 percent or more. These first order approximations should be checked with available experience.

e. Minimizing Settlement. Dynamic settlement may be insignificant provided that the sum of dynamic and static bearing stresses remain less than 1/2 of the allowable bearing capacity. Settlements that might occur under sustained dynamic loadings may be minimized by precompaction of the soil using dynamic methods. Dynamic compaction subjects the soil to severe dynamic loads that reduces the influence of any later shaking on settlement. Refer to Chapter 6 for dynamic compaction methods of minimizing settlement. Refer to ER 1110-2-1806 for general guidance and direction for seismic design and evaluation for all Corps of Engineer civil works projects.

4-2. Settlement from Earthquakes. Earthquakes primarily cause shear stress, shear strain, and shear motion from deep within the earth that propagates up toward the ground surface. This shear can cause settlement initially in deep soil layers followed by settlement in more shallow layers. Settlement caused by ground shaking during earthquakes is often nonuniformly distributed and can cause differential movement in structures leading to major damage. Settlement can occur from compaction in moist or dry cohesionless soil and from dissipation of excess hydrostatic pore pressure induced in saturated soil by earth quake ground motions. Ground motions are multidirectional; however, measurements are generally made in two horizontal and one vertical acceleration components that propagate upward from underlying rock. The vertical component of acceleration is often considered to account for less than 25 percent of the settlement, but this percentage may be exceeded. Soil affected by ground motion and subsequent settlement may extend to considerable depth depending on the source of motion.

a. Tentative Simplified Procedure for Sand. A tentative simplified procedure to estimate settlement from the shaking forces of earthquakes on saturated sands that are at initial liquefaction and on dry sands is given in Table 4-1. Input data for this procedure include the blowcount N from SPT data as a function of depth, effective and total overburden pressures σ_o' and σ_o , and an estimate of the maximum horizontal acceleration of the ground surface from earthquake records (e.g., Regulation Guide 1.6, Nuclear Regulatory Commission, items 33 and 34; Office, Chief of Engineer policy for Corps of Engineer specifications for ground motions is provided by the Earthquake Engineering and Geosciences Division, Geotechnical Laboratory, USAE Waterways Experiment Station).

(1) Application. The procedure is applied to the Tokachioki earthquake in Table 4-2.

(2) Validation. This tentative procedure has not been fully validated. The example problems in Table 4-2 are based on estimated field behaviors and not on measured data against which to validate a settlement analysis.

b. WES Procedure for Sands. The Waterways Experiment Station is currently preparing a procedure on a validated (against centrifuge test data) 2-D soil-structure interactive, nonlinear dynamic effective stress analysis which computes dynamic response histories of motions, stresses, pore water pressures, and volume changes for the range of responses and pore water pressures up to and including the initial liquefaction condition. The effects of pore water pressures on moduli, motions, stresses, and volume changes are taken into account for the entire time history of an earthquake.

4-3. Settlement from Repeated Loads and Creep. Structures subject to repeated vertical loads experience a long-term settlement from the compression of cumulative cyclic loads and secondary compression or creep. Operating machinery, pile driving, blasting, wave or wind action are common causes of this type of dynamic loading. Methods of estimating secondary compression are provided in Section IV, Chapter 3.

a. Compaction Settlement from Machine Vibrations. A procedure to estimate settlement in sand layers from machine vibrations is described in Table 4-3. The procedure is applied to an example in Table 4-4.

b. Settlement Calculated from Laboratory Cyclic Strain Tests. Drained cyclic triaxial tests may be performed on pervious soil to evaluate the cyclic settlement through a cyclic strain resistance r_ε (item 25).

(1) Test procedure. The soil should be consolidated to simulate the in situ stress state of effective horizontal and vertical pressures. The soil is subsequently subject to three different cyclic stress levels to evaluate r_ε .

Table 4-1

A Suggested Tentative Procedure for Computation of Earthquake Settlement in Sand (Data from Item 63)

a. Saturated Sand that Reaches Initial Liquefaction

Step	Description
1	Determine the blow count N from SPT tests as a function of depth and divide the profile into discrete layers of sand with each layer containing sand with similar blow count.
2	Determine correction factor C_{ER} as follows where C_{ER} = estimated rod energy in percent/60

Country	Hammer	Hammer Release	C_{ER}
Japan	Donut	Free-Fall	1.3
	Donut	Rope and Pulley with special throw release	1.12*
USA	Safety	Rope and Pulley	1.00*
	Donut	Rope and Pulley	0.75
Europe	Donut	Free-Fall	1.00*
China	Donut	Free-Fall	1.00*
	Donut	Rope and Pulley	0.83

*Prevalent method in USA today

3 Estimate the total and effective overburden pressure σ_o and σ_o' in tsf units from known or estimated soil unit weights and pore water pressures of each layer.

4 Estimate the relative density D_r in percent from results of SPT data using Figure 4-1, improved correlations for overconsolidated soil (item 50), or the expression

$$D_r = 21 \bullet \frac{N_J}{\sigma_o' + 0.7} \qquad (4\text{-}1a)$$

where N_J is the blowcount by Japanese standards and σ_o' is the effective overburden pressure. D_r for normally consolidated material may be estimated by (item 42)

$$D_r = 11.7 + 0.76 \bullet [222N + 1600 - 736\sigma_o' - 50c_u^2]^{1/2} \qquad (4\text{-}1b)$$

where

σ_o' = effective overburden pressure, tsf
c_u = uniformity coefficient, D_{60}/D_{10}
D_{60} = grain diameter at which 60 percent of soil weight is finer
D_{10} = grain diameter at which 10 percent of soil weight is finer

Table 4-1a. Continued

Step	Description
5	Determine the correction factor C_N from Figure 4-2 using σ_o' and D_r .
6	Calculate normalized blowcount $$(N_1)_{60} = C_{ER} \bullet C_N \bullet N \quad (4\text{-}2)$$ where $(N_1)_{60}$ = SPT blowcount normalized to an effective energy delivered to the drill rod at 60 percent of theoretical free-fall energy.
7	Calculate the cyclic shear stress ratio causing initial liquefaction to occur for the given earthquake of magnitude M $$\left[\frac{\tau_{av}}{\sigma_o'}\right]_M = 0.65 \bullet \frac{a_{max}}{g} \bullet \frac{\sigma_o}{\sigma_o'} \bullet r_d \quad (4\text{-}3)$$ where τ_{av} = average cyclic shear stress induced by earthquake shaking, tsf σ_o' = effective overburden pressure, tsf σ_o = total overburden pressure, tsf a_{max} = maximum horizontal acceleration of the ground surface in units of g from earthquake records of magnitude M (Regulation Guide 1.60, Nuclear Regulatory Commision; refer to Earthquake Engineering and Geosciences Division, Geotechnical Laboratory, USAE Waterways Experiment Station for Corps of Engineers) g = acceleration of gravity, 32.2 ft/sec/sec r_d = stress reduction factor; 1.0 at the ground surface decreasing to 0.9 at depth 30 ft below ground surface
8	Convert $(\tau_{av}/\sigma_o')_M$ to an equivalent earthquake of magnitude M = 7.5 by $$\left[\frac{\tau_{av}}{\sigma_o'}\right]_{7.5} = \left[\frac{\tau_{av}}{\sigma_o'}\right]_M \bullet \frac{1}{r_m} \quad (4\text{-}4)$$ where the scaling factor r_m is

Table 4-1a. Continued

Step	Description

Magnitude of Earthquake M	*No Representative Cycles at $0.65\tau_{max}$	r_m
8.5	26	0.89
7.5	15	1.00
6.75	10	1.13
6.00	5	1.32
5.25	2 - 3	1.50

* Representative of the number of equivalent stress cycles caused by the earthquake where τ_{max} = maximum cycle stress

Step	Description
9	Evaluate volumetric strain ε_c in percent after initial liquefaction from Figure 4-3 using calculated values of $(N_1)_{60}$ of step 6 and $(\tau_{av}/\sigma'_o)_{7.5}$ of step 8.
10	Evaluate earthquake settlement ρ_e after initial liquefaction in inches from $$\rho_e = \sum_{j=1}^{n} \frac{\epsilon_c}{100} \bullet h_j \qquad (4\text{-}5)$$ where h_j = thickness of each stratum j in inches

b. Dry Sand

Step	Description
1-6	Repeat steps 1 through 6 in Table 4-1a above to evaluate D_r and $(N_1)_{60}$.
7	Evaluate mean effective pressure σ'_m of each stratum in tsf (e.g., $\sigma'_m = \frac{(1 + 2K_o)}{3} \bullet \sigma'_o = 0.65\sigma'_o$ if the coefficient of lateral earth pressure $K_o = 0.47$) . σ'_m is considered the total mean pressure in dry sand.
8	Calculate $$G_{max} = 10 \bullet [(N_1)_{60}]^{1/3} \bullet [\sigma'_m]^{1/2} \qquad (4\text{-}6)$$ where G_{max} = maximum shear modulus, tsf

Table 4-1a. Concluded

Step	Description
9	Evaluate using G_{max} from step 8 $$\gamma_{eff} \bullet \frac{G_{eff}}{G_{max}} = \frac{0.65 \bullet a_{max} \bullet \sigma_o \bullet r_d}{g \bullet G_{max}} \quad (4\text{-}7)$$ where γ_{eff} = effective cyclic shear strain induced by an earthquake G_{eff} = effective shear modulus at earthquake induced shear stress, tsf a_{max} = maximum horizontal acceleration at the ground surface in units of g σ_o = total overburden pressure, tsf g = acceleration of gravity, 32 ft/sec/sec r_d = stress reduction factor; 1.0 at the ground surface decreasing to 0.9 at a depth 30 ft below ground surface
10	Evaluate γ_{eff} from Figure 4-4 using $\gamma_{eff} \cdot G_{eff}/G_{max}$ from step 9 and σ'_m from step 7; multiply by 100 to convert to percent.
11	Use γ_{eff} and evaluate volumetric strain in percent $\varepsilon_{c,7.5}$ from Figure 4-5 using D_r or $(N_1)_{60}$ for an M = 7.5 magnitude earthquake.
12	Evaluate volumetric strain ratio from Figure 4-6 for the given magnitude of earthquake M and multiply this ratio by $\varepsilon_{c,7.5}$ to calculate $\varepsilon_{c,M}$.
13	Multiply $\varepsilon_{c,M}$ by 2 to consider the multidirection effect of earthquake shaking on settlement and evaluate total earthquake induced settlement of each stratum j of thickness h_j for n strata by $$\rho_e = \sum_{j=1}^{n} \frac{2\epsilon_{c,M}}{100} \bullet h_j \quad (4\text{-}8)$$

Table 4-2

Example Applications of Simplified Procedure
to Estimate Earthquake Settlement

a. Saturated Sand at Initial Liquefaction Condition

Layer	Thickness, ft	N	C_{ER}	σ'_o, psf	C_N	$(N_1)_{60}$	$\frac{\tau_{av}}{\sigma_o}$	ε_c, percent	ρ_e, in.
1	4.0	1.0	0.82*	240					
2	3.3	0.5	0.82*	575	1.7	0.7	0.155	10.0	4.0
3	3.3	0.5	0.82*	764	1.57	0.6	0.185	10.0	4.0
4	3.3	0.5	1.09	954	1.44	0.8	0.200	10.0	4.0
5	3.3	2.0	1.09	1144	1.34	2.9	0.210	5.5	2.2
6	3.3	5.0	1.09	1334	1.24	6.8	0.215	3.2	1.3
7	3.3	23.0	1.21	1523	1.16	32.0	0.220	0.0	0.0
8	3.3	33.0	1.21	1713	1.09	44.0	0.225	0.0	
9	3.3	28.0	1.21	1903	1.03	35.0	0.225	0.0	
10	3.3	33.0	1.21	2093	0.97	39.0	0.225	0.0	

Note: *Corrected by 0.75 — Total Settlement = 15.5
Water Table = 4 ft — Observed Maximum Settlement ≈ 20
Estimated Maximum Acceleration $a_{max} = 0.2$ g

b. Dry Sand

Layer	Thickness, ft	σ'_o, psf	G_{max}, ksf	$\gamma_{eff} \frac{G_{eff}}{G_{max}}$	γ_{eff}	$\varepsilon_{c,7.5}$, percent	$\varepsilon_{c,6.6}$, percent	ρ_e, in.
1	5	240	520	0.00013	0.0005	0.14	0.11	0.13
2	5	715	900	0.00023	0.0008	0.23	0.18	0.22
3	10	1425	1270	0.00032	0.0012	0.35	0.28	0.67
4	10	2375	1630	0.00040	0.0014	0.40	0.32	0.77
5	10	3325	1930	0.00045	0.0015	0.45	0.36	0.86
6	10	4275	2190	0.00046	0.0013	0.38	0.30	0.71

Note: Total estimated settlement = 2.7 inches — Total Settlement = 3.37

$D_r = 45$ percent — $(N_1)_{60} = 9$
$a_{max} = 0.45$ g

Reprinted by permission of the American Society of Civil Engineers from Journal of Geotechnical Engineering, Vol 118, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", by K. Tokimatsu and H. B. Seed, p. 871, 876

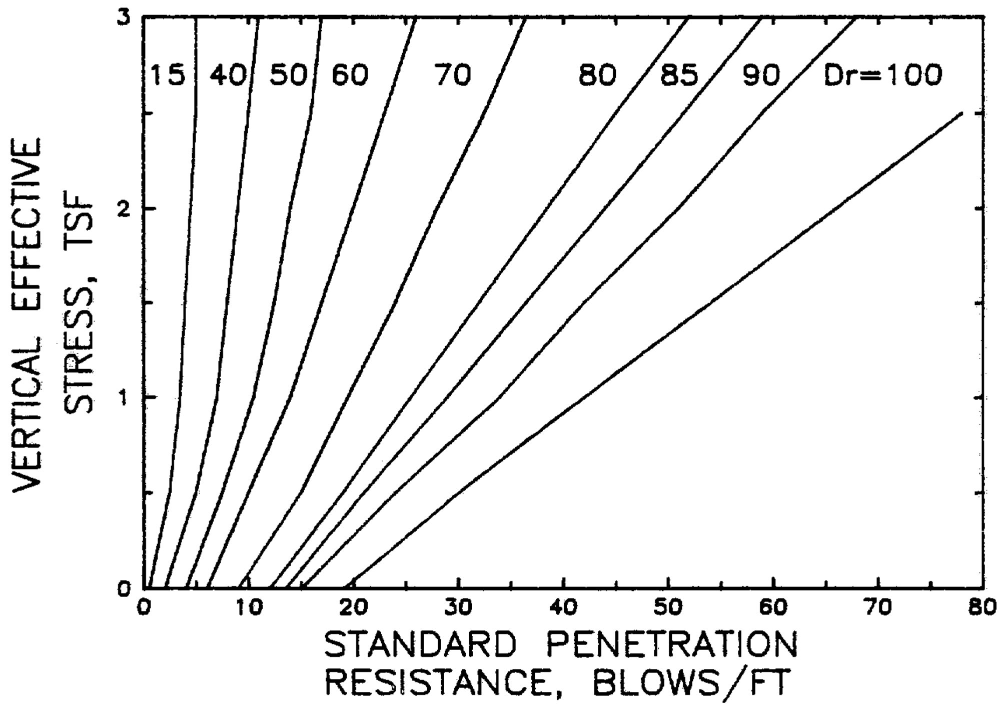


Figure 4-1. Correlations between relative density and blow count N from SPT after Gibbs and Holtz (data from NAVFAC DM-7.1)

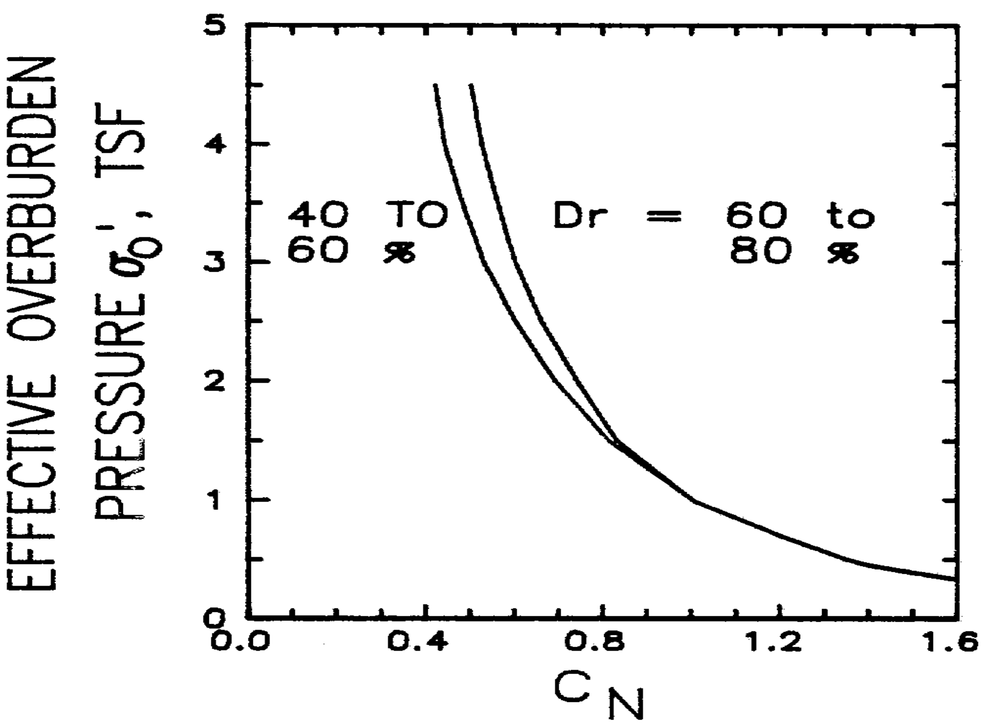


Figure 4-2. Curves for determination of C_N (data from item 62)

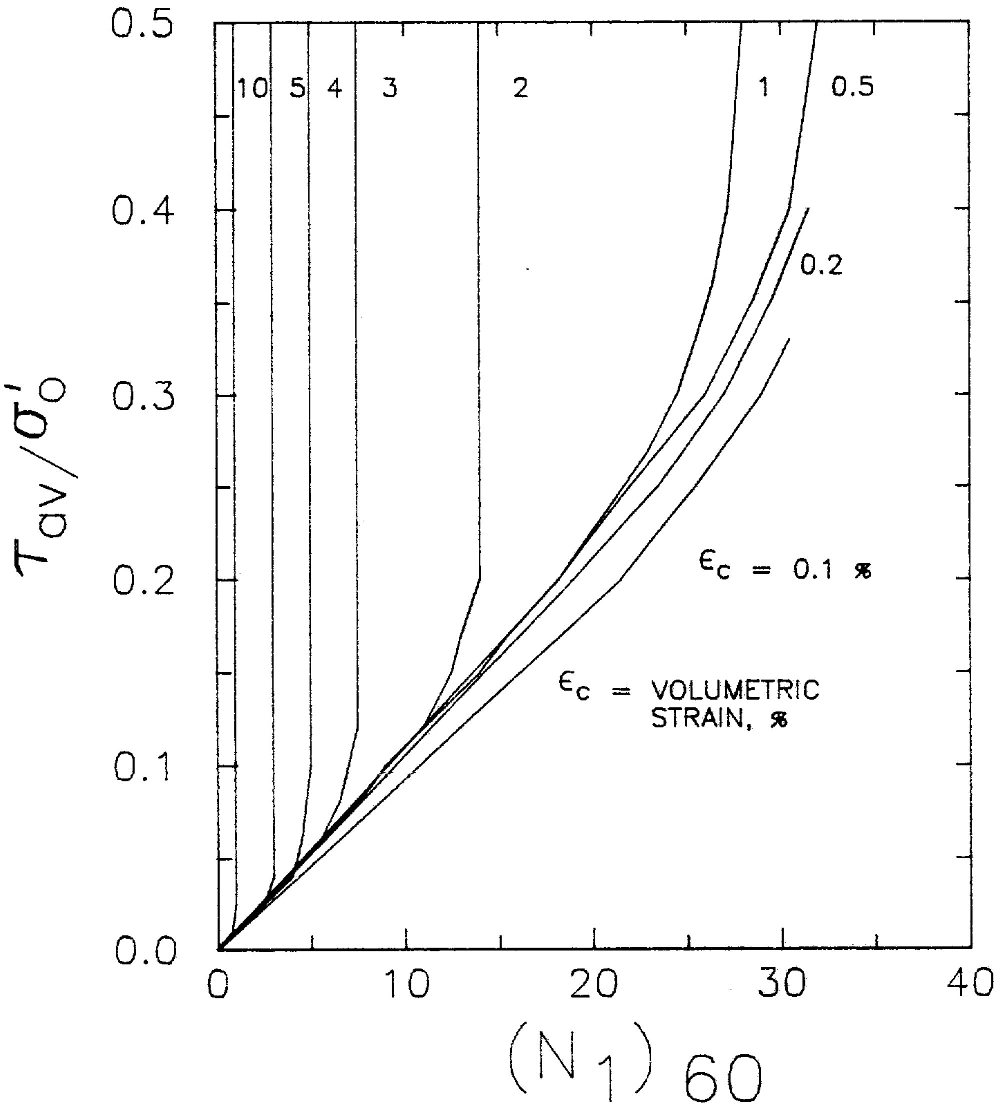


Figure 4-3. Proposed relationship between cyclic stress ratio $(N_1)_{60}$ and volumetric strain ε_c for saturated clean sands for M = 7.5 earthquake. Reprinted by permission of the American Society of Civil Engineers from Journal of Geotechnical Engineering, Vol 118, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", by K. Tokimatsu and H. B. Seed, p. 866.

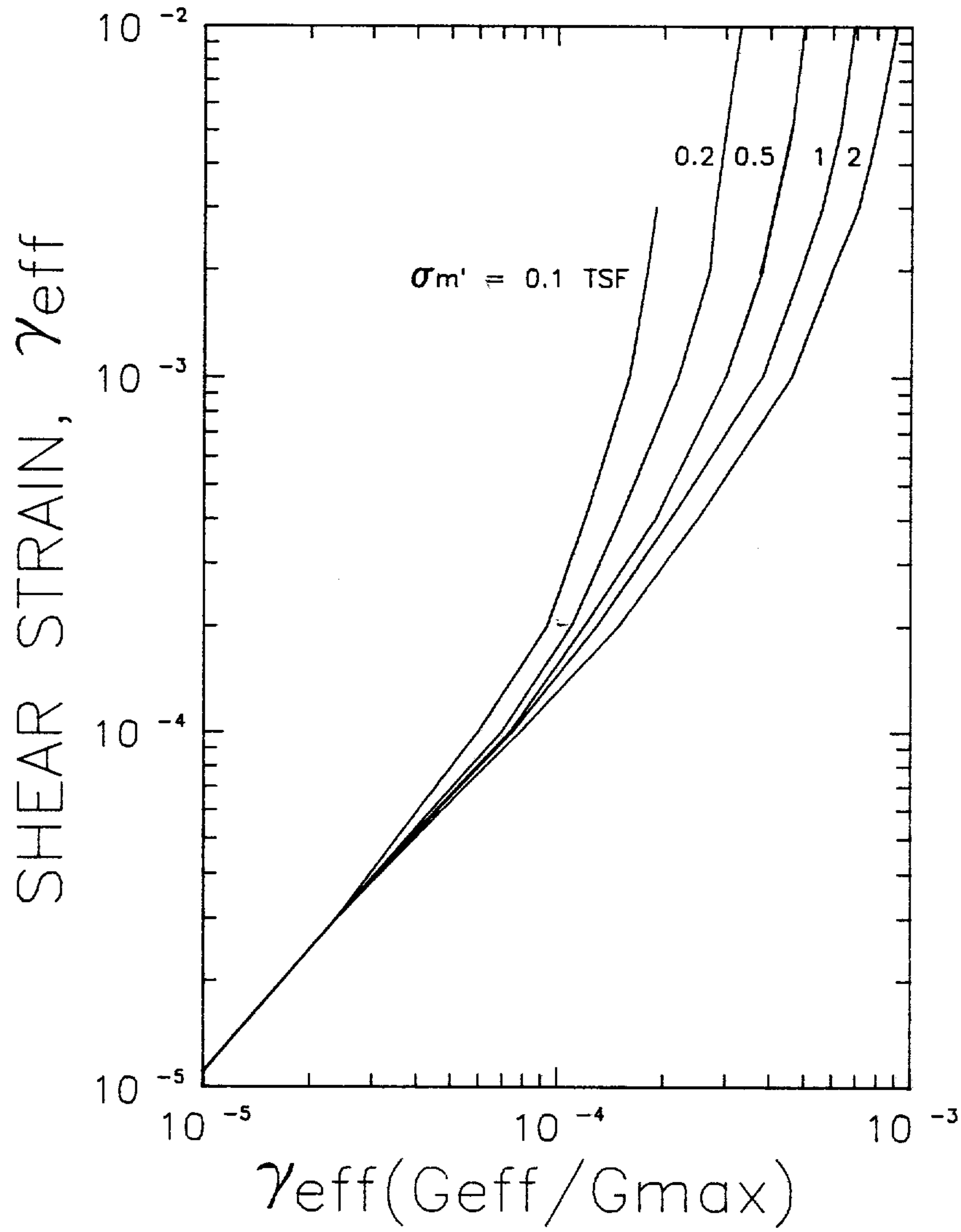


Figure 4-4. Plot for determination of induced strain in sand deposits. Reprinted by permission of the American Society of Civil Engineers from Journal of Geotechnical Engineering, Vol 118, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", by K. Tokimatsu and H. B. Seed, p. 873.

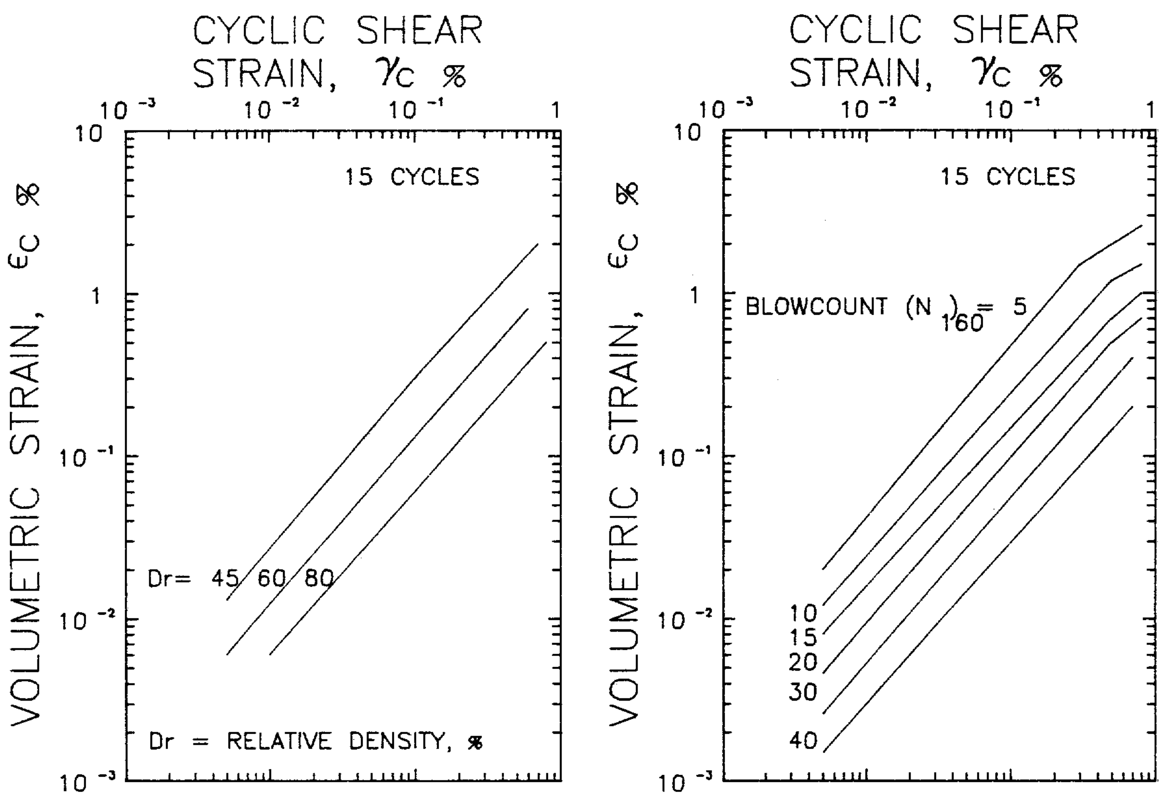


Figure 4-5. Relationships between volumetric strain ε_c and cyclic shear strain γ_c for dry sand and earthquake magnitude M = 7.5. Reprinted by permission of the American Society of Civil Engineers from Journal of Geotechnical Engineering, Vol 118, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", by K. Tokimatsu and H. B. Seed, p. 874.

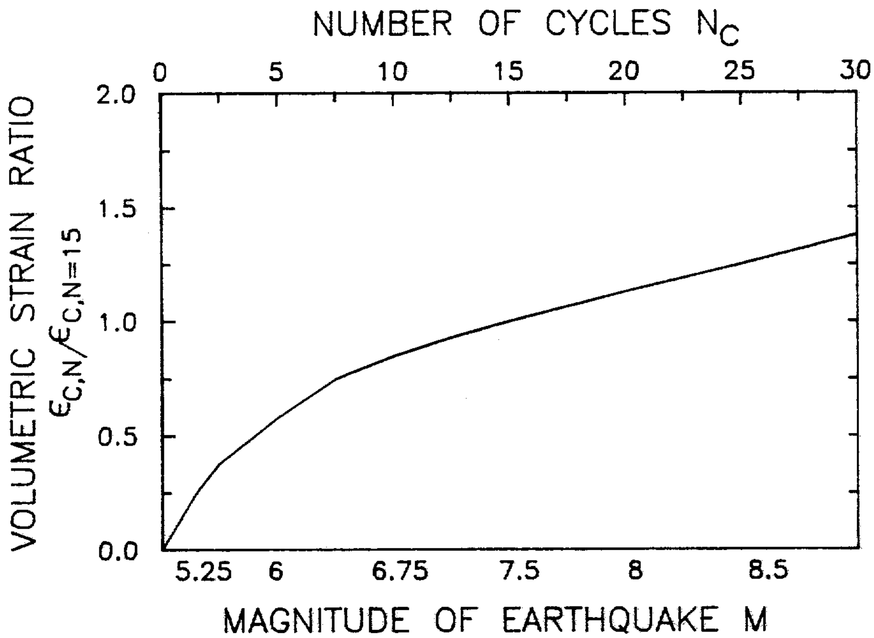


Figure 4-6. Relationship between volumetric strain ratio and number of cycles (earthquake magnitude) for dry sands. Reprinted by permission of the American Society of Civil Engineers from Journal of Geotechnical Engineering, Vol 118, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking", by K. Tokimatsu and H. B. Seed, p. 874.

(a) The soil should be consolidated so that a plot of one-half of the deviator stress versus the effective horizontal confining pressure provides a slope indicative of a realistic effective coefficient of lateral earth pressure. The slope s of this curve required to obtain a given coefficient of lateral earth pressure K_o is

$$s = 0.5 \bullet \left(\frac{1}{K_o} - 1\right) \tag{4-9}$$

For example, the slope s should be 0.7 if K_o is 0.42. The soil should be consolidated to an effective horizontal confining pressure simulating the in situ soil.

(b) Additional vertical dynamic loads should be applied so that the soil specimen is subject to three different cyclic stress levels of 200 to 300 cycles per stress level. The effective lateral confining pressure is maintained constant.

(c) The cumulative strain as a function of the number of cycles N at each stress level should be plotted as shown in Figure 4-7a. The slope of the curves in Figure 4-7a is the strain resistance $R_\varepsilon = dN/d\varepsilon$.

(d) The strain resistance should be plotted versus the number of cycles as shown in Figure 4-7b for each stress level. A straight line should subsequently be plotted through these data points for each stress level. The slope of this line is the cyclic strain resistance r_ε .

Table 4-3

Settlement From Machine Vibrations
(After NAVFAC DM-7.3)

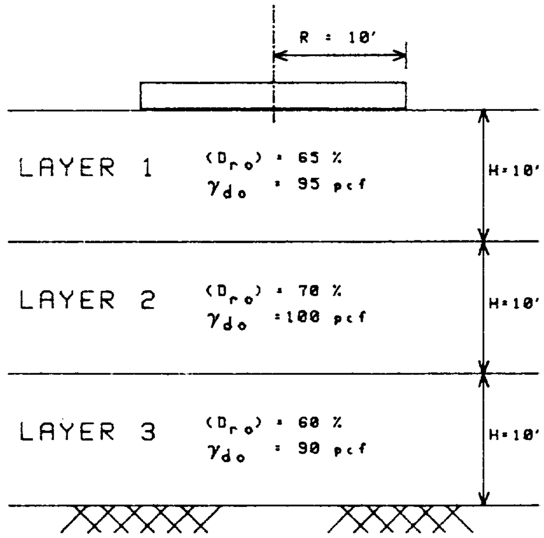
Step	Description
1	Evaluate initial relative density D_{roj} of each soil layer j from the blow count N by Equations 4-1, Figure 4-1, or improved correlations (item 50).
2	Estimate or measure maximum displacement of vibration Amax in inches and frequency of vibration f in revolutions per minutes at the base of the foundation.
3	Calculate the frequency of vibration in radians per second from $$\omega_o = \frac{2\pi f}{60} \qquad (4\text{-}10)$$
4	Calculate acceleration of vibrations in g at foundation level a_o $$a_o = \frac{\omega_o^2 \bullet Amax}{12 \bullet 32.2} \qquad (4\text{-}11)$$
5	Calculate a_j acceleration of vibration in g at midpoint of each soil layer j by $$d_{mj} > R: \quad a_j = a_o \bullet \left[\frac{R}{d_m}\right]^{1/2} \qquad (4\text{-}12a)$$ $$d_{mj} \le R: \quad a_j = a_o \qquad (4\text{-}12b)$$ where d_{mj} = distance from foundation base to midpoint of soil layer j , ft R = equivalent radius of foundation $\sqrt{LB/\pi}$, ft
6	Calculate the critical acceleration in g of each soil layer j $$a_{critj} = \frac{-\ln \bullet \left[1 - \frac{D_{roj}}{100}\right]}{\beta_v} \qquad (4\text{-}13)$$ where D_{roj} = initial relative density at zero acceleration of layer j , percent β_v = coefficient of vibratory compaction β_v depends on water content W in percent and varies approximately

Table 4-3. Concluded

	$W < 5\ percent$: $\beta_v = 0.2 + 0.12W$
	$5 \le W \le 18\ percent$: $\beta_v = 0.77 + 0.006W$
	β_v decreases if the water content is greater than 18 percent.
7	Estimate the final relative density D_{rfj} of each layer j from $a_j > a_{critj}$ $\quad D_{rfj} = 100 \bullet \left[1 - e^{-\beta_v \bullet (a_{critj} + a_j)}\right]$ (4-14a) $a_j \le a_{critj}$ $\quad D_{rfj} = D_{roj}$ (4-14b)
8	Calculate the change in relative density ΔD_{jr} of each soil layer j by $\Delta D_{rj} = D_{rfj} - D_{roj}$ (4-15)
9	Calculate the settlement in feet of each soil layer j by $\rho_{vj} = 0.0025 \bullet \frac{\Delta D_{rj}}{100} \bullet \gamma_{do} \bullet H_j$ (4-16) where γ_{do} = initial dry density of the sand layer, lbs/ft^3 H_j = stratum of thickness, ft Equation 4-16 is based on the range of maximum and minimum dry densities for sands reported in item 6
10	Add the settlements of each layer to find the total settlement.

Table 4-4

Example Calculation for Vibrations Induced Compaction Settlement Under Operating Machinery (From NAVFAC DM-7.3)



GIVEN: SOIL PROFILE AS SHOWN, FOOTING WITH A RADIUS OF 10 FEET SUBJECTED TO A VIBRATORY LOAD CAUSING A PEAK DYNAMIC DISPLACEMENT Amas = 0.007 IN. OPERATING FREQUENCY f = 2500 RPM. WATER CONTENT W = 16 %, USE $\beta_v = 0.88$

$$\omega_o = \frac{2500}{60}\cdot 2\pi = 261.8 \text{ RAD/SEC}$$

$$a_o = \frac{(261.8)^2\cdot 0.007}{12\cdot 32.2} = 1.24g$$

LAYER 1: DEPTH TO MID LAYER $d_m = 5' < R = 10'$

USE $a_j = a_o = 1.24g$

$$a_{critj} = \frac{-\ln\left[1 - \frac{D_{roj}}{100}\right]}{\beta_v}$$

$$a_{critj} = \frac{-\ln\left[1 - \frac{65}{100}\right]}{0.88} = 1.19g$$

$a_j = 1.24g > a_{critj}$

USE $D_{rfj} = 100\left[1 - e^{-\beta_v\cdot\left[a_{critj} + a_j\right]}\right]$

$$= 100\left[1 - e^{-0.88\cdot(1.19 + 1.24)}\right]$$

$$= 100\cdot(1 - .118) = 88\ \%$$

$$\Delta D_{rj} = D_{rfj} - D_{roj} = 88 - 65 = 23\ \%$$

$$\rho_{vj} = 0.0025\cdot\frac{\Delta D_{rj}}{100}\cdot\gamma_{do}\cdot H_j$$

$$= 0.0025\cdot 0.23\cdot 95\cdot 10 = 0.55 \text{ FT OR } 6.6 \text{ IN.}$$

LAYER 2: DEPTH TO MIDLAYER $d_m = 15'$

$$a_j = a_o\cdot\left[\frac{R}{d_m}\right]^{1/2} = 1.24\cdot\left[\frac{10}{15}\right]^{1/2} = 1.01g$$

$$a_{critj} = \frac{-\ln(1 - 0.7)}{0.88} = 1.37g$$

$a_i = 1.01g < a_{critj}$

$D_{rj} = D_{roj} = 70\ \%$

NO SIGNIFICANT COMPACTION SETTLEMENT LIKELY

LAYER 3: DEPTH TO MID LAYER $d_m = 25'$

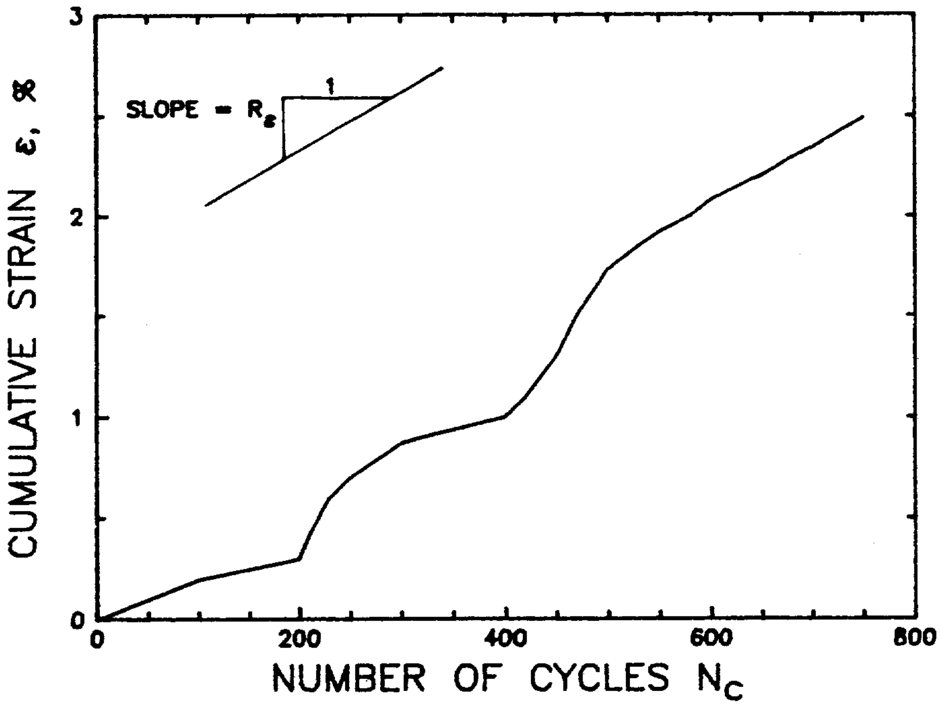
$$a_i = 1.24\cdot\left[\frac{10}{25}\right]^{1/2} = 0.78g$$

$$a_{critj} = \frac{-\ln(1 - 0.6)}{0.88} = 1.04g$$

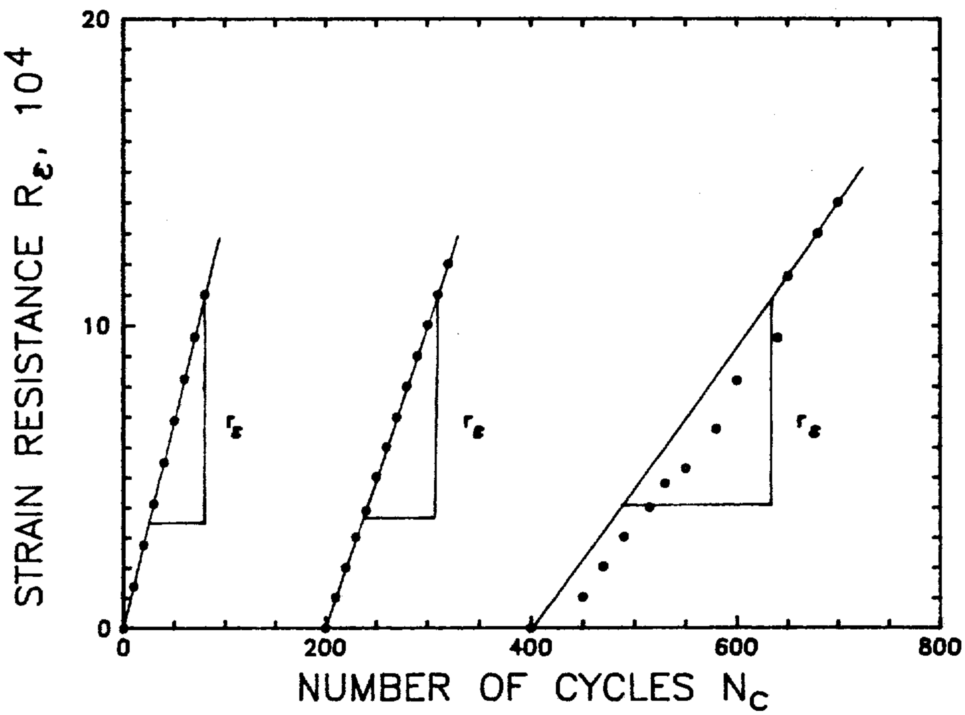
$a_j < a_{critj}$

NO SIGNIFICANT COMPACTION SETTLEMENT LIKELY

ANTICIPATED COMPACTION SETTLEMENT = 6.6 IN. RELATIVE DENSITY OF TOP LAYER SHOULD BE INCREASED TO 70 % OR GREATER



a. CUMULATIVE STRAIN VERSUS NUMBER OF CYCLES



b. STRAIN RESISTANCE VERSUS NUMBER OF CYCLES

Figure 4-7. Example of strain and strain resistance as a function of cycles N_c for different stress levels

(e) The cyclic strain resistance decreases with increasing stress levels and approaches zero when the shear strength is fully mobilized. The cyclic strain resistance may increase with increasing depth because the percentage of mobilized shear strength may decrease with increasing depth.

(2) Calculation of settlement. The settlement of a pervious layer of thickness H caused by repeated loads may be given for this drained soil by (item 25)

$$\rho_r = \frac{H}{r_\epsilon} \bullet \ln N \tag{4-17}$$

where

ρ_r = settlement of the layer from repeated load, ft
H = thickness of stratum, ft
r_ε = cyclic strain resistance of stratum from laboratory tests
N = number of cycles of repeated load

The appropriate value of r_ε to select from the laboratory test results depends on the maximum anticipated stress level in the soil caused by the repeated loads. For example, the maximum anticipated stress in the soil level may be calculated from the exciting force by methods in Appendix C. The exciting force may be calculated from guidance provided in NAVFAC DM-7.3.

(3) Alternative settlement calculation. An alternative method of evaluating effects of repeated loads on settlement of clayey soil from laboratory cyclic triaxial tests is to apply the creep strain rate formulation (item 24)

$$\epsilon_t = \epsilon_{t1} + \frac{e^\alpha}{1 - \lambda_d} \bullet \left[t^{1-\lambda_d} - t_1^{1-\lambda_d}\right] \tag{4-18a}$$

If $\lambda_d = 1$, then

$$\epsilon_t = \epsilon_{t1} + e^\alpha \bullet \ln\frac{t}{t_1} \tag{4-18b}$$

where

ε = strain at time t
ε_{t1} = strain at time t_1 or after one cycle
e = base e or 2.7182818
α = $C \cdot \sigma_{rd} - B$
B, C = constants from Table 4-5
σ_{rd} = repeated deviator stress, tsf
λ_d = decay constant found from slope of logarithmic strain rate $\varepsilon_N/\varepsilon_1$ versus logarithm number of cycles N_c, Figure 4-8,

(a) Settlement may be found by substituting ε_t of Equation 4-18b for ε_c of Equation 4-5, Table 4-1a. Evaluation of ε_t from Equations 4-18 is appropriate for repeated loads with frequencies between 0.1 Hz and 10 Hz, a typical range for traffic loads; however, settlement may be underestimated because traffic loads are more complex than compressive vertical loads. Repeated loads with various periods and rest intervals between repeated loads do not appear to cause significant change in strain.

Table 4-5

Constants B and C to Evaluate Creep Constant α as a Function of Overconsolidation Ratio OCR (Data from Item 24)

OCR	C	B
4	3.5	9.5
10	2.8	9.2
20	3.7	9.5

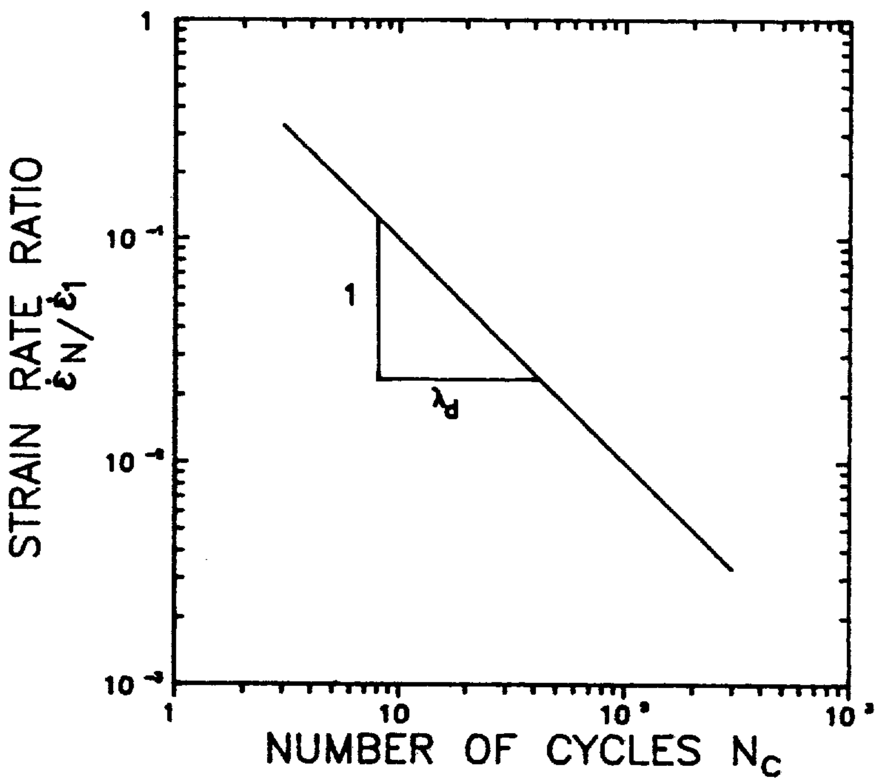


Figure 4-8. Example decay constant

(b) An application of Equations 4-18 to London clay where $\lambda_d = 1$, $\varepsilon_{t1} = 0.0$ at $t_1 = 1$ second , $\sigma_{rd} = 1$ tsf , and OCR = 4 is

$$\epsilon_t = \epsilon_{t1} + e^{\alpha} \bullet \ln\frac{t}{t_1} = e^{(3.5 \bullet 1.0 - 9.5)} \bullet \ln t$$

$$= \frac{\ln t}{e^6} = \ln t/403.4 = 0.0025 \ln t$$

After 10 seconds the strain ε_{10} is 0.0058. Settlement is the strain times thickness of the stratum.