ARKANSAS BRIDGE FOUNDATIONS

DESIGN AND ANALYSIS

Arkansas

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FINAL REPORT
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16. Abstract

The fifth and final report in a series of reports that detail field exploration, laboratory testing and design and analysis of bridge foundations in Arkansas. Recommended procedures for design and analysis of bridge foundations are the substance of this report. The procedures cover both shallow and deep foundations.

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DESIGN AND ANALYSIS

by

Robert C. Welch

Sam I. Thornton

FINAL REPORT

HIGHWAY RESEARCH PROJECT 39

conducted for

The Arkansas State Highway and Transportation Department

in cooperation with

The U.S. Department of Transportation

Federal Highway Administration

SEPTEMBER, 1978

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SUMMARY

This report is the fifth and final report in a series of reports to provide the Arkansas State Highway and Transportation Department with detailed procedures for field exploration, laboratory testing, and design and analysis of bridge foundations.

Recommended procedures for design and analysis of bridge foundations are the substance of this report. The procedures cover both shallow and deep foundations.

Shallow foundations may fail from loss of bearing capacity or excessive settlement. Examples of foundation design to prevent loss of bearing capacity are included. Bearing capacity examples include the effect of: a) footing shape, b) eccentric load, c) inclined load, d) ground surface slope, e) water table, and f) compressibility. An example for estimation of the settlement due to primary consolidation is also given.

Deep foundations, including piles, drilled shafts, and caissons, get their load capacity from friction and/or adhesion on the foundation sides and bearing on the end of the foundation. Deep foundation capacity may be estimated by: a) methods based on soil properties, b) pile driving formula, or c) load tests. Examples of foundation design based on soil properties and pile driving formula are included.

Procedures for site investigation, laboratory testing, and design and analysis of bridge foundations are recommended for adoption by the Arkansas State Highway and Transportation Department.

GAINS, FINDINGS, AND CONCLUSIONS

Four recommendations, listed below, are suggested for adoption by the Arkansas State Highway and Transportation Department.

1. The general bearing capacity equation, with appropriate corrections for compressibility, shape, eccentricity, inclination of load, ground surface slope, and position of the water table should be used for determining the ultimate load capacity of shallow foundations.
2. Settlement should be estimated for each footing. The analysis of settlement of footings on cohesive soils should be based upon consolidation test results. Settlement of footings on cohesionless soils may be based upon empirical correlations.
3. Preliminary estimates of pile capacity should be done by the limit equilibrium method. These estimates should be verified at the time of installation by wave equation analyses or by a comprehensive dynamic formula such as the Hiley formula. Pile load tests should be performed on large jobs and in difficult soil conditions; and the results correlated with the limit equilibrium analysis and driving resistance.
4. The immediate settlement of piles should be estimated by the load transfer function approach if no pile load test has been performed. Long term consolidation settlement should be estimated by using the Mindlin solution to determine stresses and one-dimensional consolidation test results to estimate the settlement.

IMPLEMENTATION STATEMENT

The following procedures are recommended for implementation by the Arkansas State Highway and Transportation Department.

SITE INVESTIGATION

1. It is recommended that AHTD develop and maintain a comprehensive file of soil data for existing and planned bridge sites. This file should contain not only data generated by AHTD but also soil maps, geologic maps, and soil data from geotechnical consultants and other state and federal government agencies.
2. Preliminary field investigations should include seismic and resistivity surveys, wherever practical, as well as preliminary borings. The signal-enhancement type of seismograph is recommended.
3. The primary objectives of the detailed field investigation should be to define the soil stratification and to obtain high quality undisturbed samples of the foundation soil. The sampling tools recommended are the Shelby tube sampler, the Osterberg piston sampler, and either the Denison sampler or the Pitcher sampler.
4. If high quality undisturbed samples cannot be obtained, the in-situ properties should be assessed by using the quasi-static cone penetrometer. For soft to very soft clays, the field vane test is also appropriate.
5. For long term measurements of water levels or pore pressures, a double tube open system piezometer is recommended.

LABORATORY TESTING

1. The tests recommended for soil classification are the liquid limit, plastic limit and particle size analysis tests. The wet preparation procedure should be used for any soil containing clay.
2. Triaxial compression tests should provide the primary means to determine shear strength. For an undrained analysis involving homogeneous, intact, saturated clay, unconfined compression tests are acceptable.
3. To assess the stability of embankments constructed on clay shale, repeated direct shear tests should be performed on the clay shale and the residual strength used in the analysis.

CHAPTER I

INTRODUCTION

This report is the fifth and final report of a series of reports to provide the Arkansas State Highway and Transportation Department with detailed procedures for field exploration, laboratory testing, and design and analysis of bridge foundations. The reports in this series are:

Arkansas Bridge Foundations: Field Investigation

This report is a summary of the state-of-the-art of field investigation procedures which may be useful in developing data for bridge foundation design.

Arkansas Bridge Foundations: Laboratory Investigation

This report is a summary of the state-of-the-art of laboratory procedures which may be useful in developing data for bridge foundation design.

Arkansas Bridge Foundations: Laboratory Procedures

This report contains detailed laboratory procedures recommended for use by the Arkansas State Highway and Transportation Department. Several types of laboratory equipment were evaluated and in some cases, different makes were compared.

Arkansas Bridge Foundations: Field Procedures

This report contains detailed procedures for performing field tests and obtaining samples recommended for use by the Arkansas State Highway and Transportation Department. The evaluations of a quasi-static cone penetrometer and geophysical exploration methods are given in appendices to this report.

Included in this fifth report are details of recommended procedures for design and analysis of bridge foundations. Chapter II presents a design and analysis procedure for shallow foundations and numerous examples of design for the various loading conditions and geometries which may be encountered. Several procedures for the design and analysis of deep foundations are presented in Chapter III. Examples of the application of some of these pro-

cedures are given. The final chapter of this report includes recommendations for implementation of the results of this project.

Specific recommendations cannot be made in some areas because each job site and soil exploration program presents a unique set of conditions. Two of these areas are the determination of the types of laboratory tests to be perfomed and the selection of the appropriate factor of safety.

Laboratory tests should be selected to duplicate, as closely as possible, the field loading conditions. The most critical case which is likely to occur should govern the design. For most foundations, the end of construction and first application of load is most critical and corresponds to undrained loading for clay soils. The appropriate laboratory test to duplicate this loading would be the unconsolidated-undrained (UU) test. For sandy soils, because of their greater permeability, the drained (CD) test would be appropriate. Judgement is required in selecting the test procedure appropriate for silty materials. For stage construction or preloading, the consolidated-undrained (CU) analysis is appropriate for clays, and for long-term stability the drained (CD) analysis should be selected.

The safety factor should reflect the confidence of the design engineer in the data being used. If a test such as the standard penetration test is performed in a somewhat careless manner, then a high safety factor would be required when using the data. Generally, if a test procedure accurately measures the property desired and the test is performed with care, a safety factor of two will be adequate. However, some test procedures give results that are only approximately correct, and therefore, safety factors of three or more are required. Approximate design procedures also generally require higher factors of safety than the more precise methods.

CHAPTER II

SHALLOW FOOTINGS

Failure of shallow footings results from two causes, settlement and loss of bearing capacity. Settlement can take place rapidly as in sand which will often settle while construction is in progress or slowly as in clay. The 183 ft. tower of Pisa, a successful failure, has been settling since its construction in 1174-1350. Loss of bearing capacity usually occurs rapidly and may result in destruction of the structure. When loss of bearing capacity occurs, large footing movements, as much as the footing width, take place. The eight concrete silos which failed and were reported by Tschebotarioff (1973) are an example (Figure 2-1).

BEARING CAPACITY

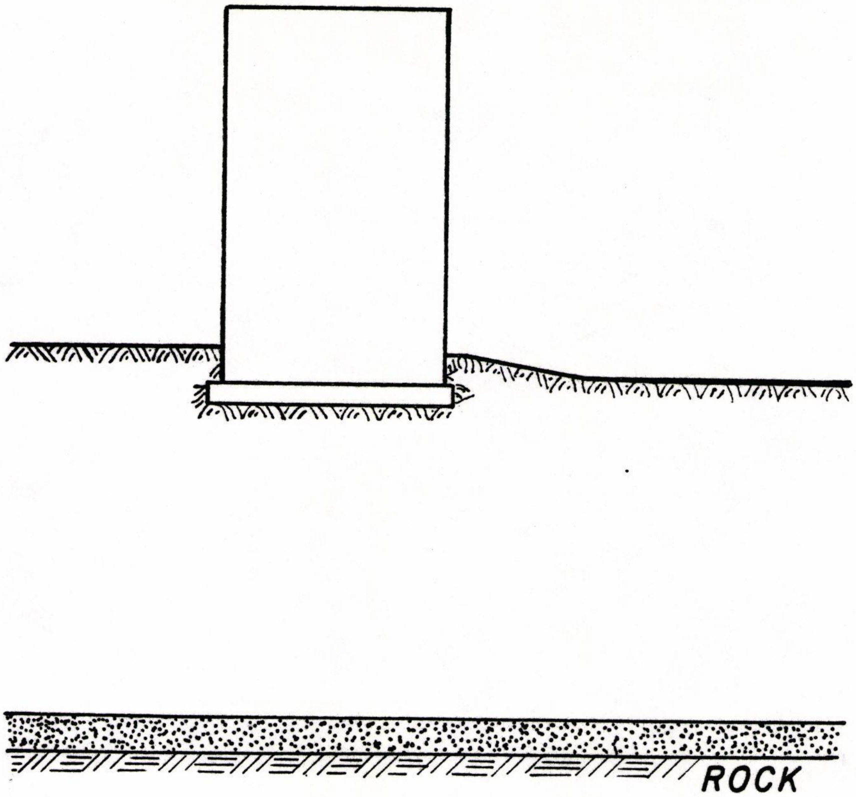
Analysis for bearing capacity, or stability, is an analysis of shear failure within the soil mass. Shear failure, depending on the soil characteristics, may take three forms: punching shear, local shear, or general shear (Figure 2-2).

Punching shear is characterized by vertical movement of the footing without tilt and little movement of soil along the sides of the footing (Figure 2-2c). The soil beneath the footing is forced down in a wedge and the soil below is forced down. Load must be increased as the failure takes place in order to maintain vertical movement. Compressible sands are likely to fail in punching shear.

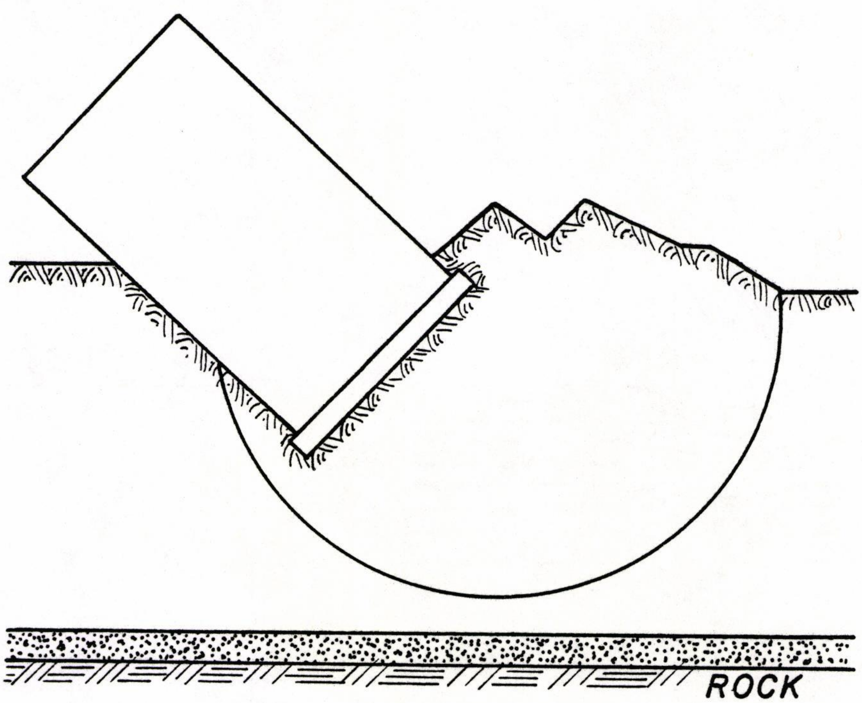
Local shear is often the failure pattern of compressible soils that can endure large strains without plastic flow. The failure pattern has a wedge and slip surfaces which start at the edge of the footing. Some soil bulging is visible on the sides of the footing, but, because the soil is compressible, the slip failure planes never appear at the surface (See dotted lines in Figure 2-2b). An increasing load is required to continue settlement in a local shear failure.

General shear is an extension of local shear and occurs in more rigid soils. The shear zone propagates outward until a well defined failure plane reaches the surface. Soil heave within the failure zone is apparent in a general shear failure. Once a failure stress is reached, settlement continues even if the stress is reduced slightly (Figure 2-2a). Because of the continued settlement with no increase in load, general shear failures are rapid and usually result in destruction of the structure.

The type of failure depends on the relative density or compressibility of the soil and depth of footing. In sand, the greater the relative density, the more likely general shear is to



Original Profile



After Bearing Capacity Failure

Figure 2-1. After Tschebotarioff, 1973.

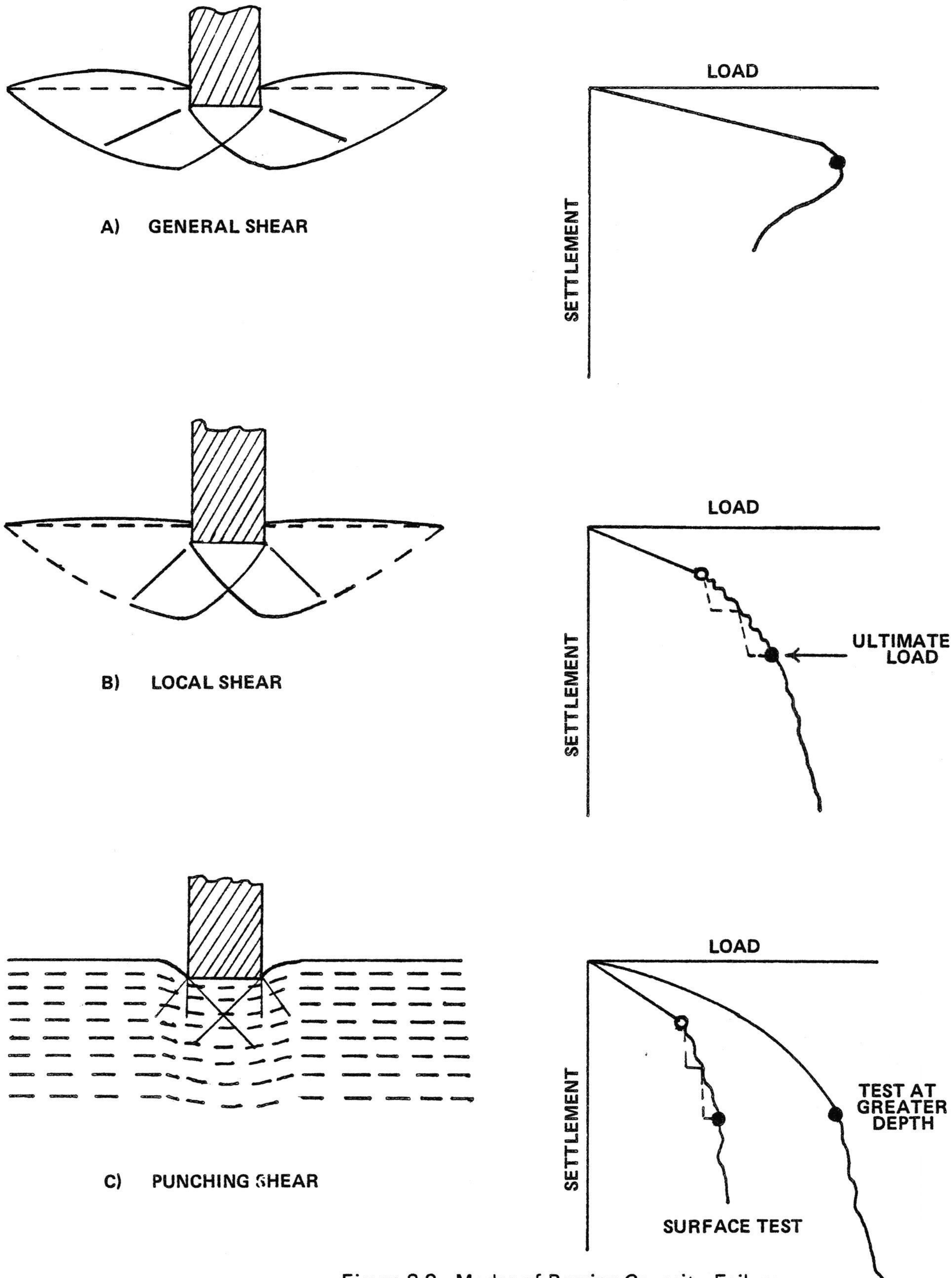


Figure 2-2. Modes of Bearing Capacity Failure.

develop (Figure 2-3). As relative density is reduced or the depth of footing increases, the more likely local shear or punching shear is to occur.

Differences between general shear, local shear, and punching shear are reasonably well understood but no general criteria exists for prediction of the failure mode. Vesic (1973) has proposed the rigidity index, I_r as a relative measure of compressibility. Application of this criteria is discussed later in the section titled "compressibility".

No exact solution for failure of soil in general shear exists (Meyerhof, 1955) because assumptions must be made to simplify the problem. An early solution for a surface footing was made in 1920 by Prandtl but the soil was assumed to be weightless and the footing perfectly rough. Hencky solved a similar problem for weightless soil with no friction on the base in 1923. Reissner (1924) improved Prandtl's solution by placing the footing below the surface. At present, bearing capacity can be found by assuming the soil is weightless as far as the influence of cohesion is concerned and by assuming the soil has no cohesion when evaluating the effect of weight.

A simple and conservative solution for the bearing capacity of a strip footing in general shear has developed (credits to Bell, Buisman, Terzaghi, and Meyerhof) from earlier exact solutions:

$$q_o = \frac{\gamma B}{2} N_\gamma + c\, N_c + q\, N_q \qquad \text{(Equation 2-1)}$$

where:
- q_o is ultimate bearing capacity
- γ unit weight
- B footing width
- c cohesion
- q surcharge load
- N_γ, N_c, N_q bearing capacity factors which depend on the angle of internal friction (Table 2-1)

To apply this solution, the real problem must be simplified to that of a surface footing with a surcharge load at its sides (Figure 2-4). This assumption is conservative because it neglects shear strength which may exist in soil above the footing. In many cases, neglecting this strength is justified because of soil disturbance during construction.

Example 1: A long footing, three feet wide, is placed at the ground surface. Find the ultimate bearing capacity for a general shear failure if the soil has an angle of internal friction of 20 degrees, cohesion of 300 psf, and unit weight of 100 pcf. Use the bearing capacity factors suggested by Vesic.

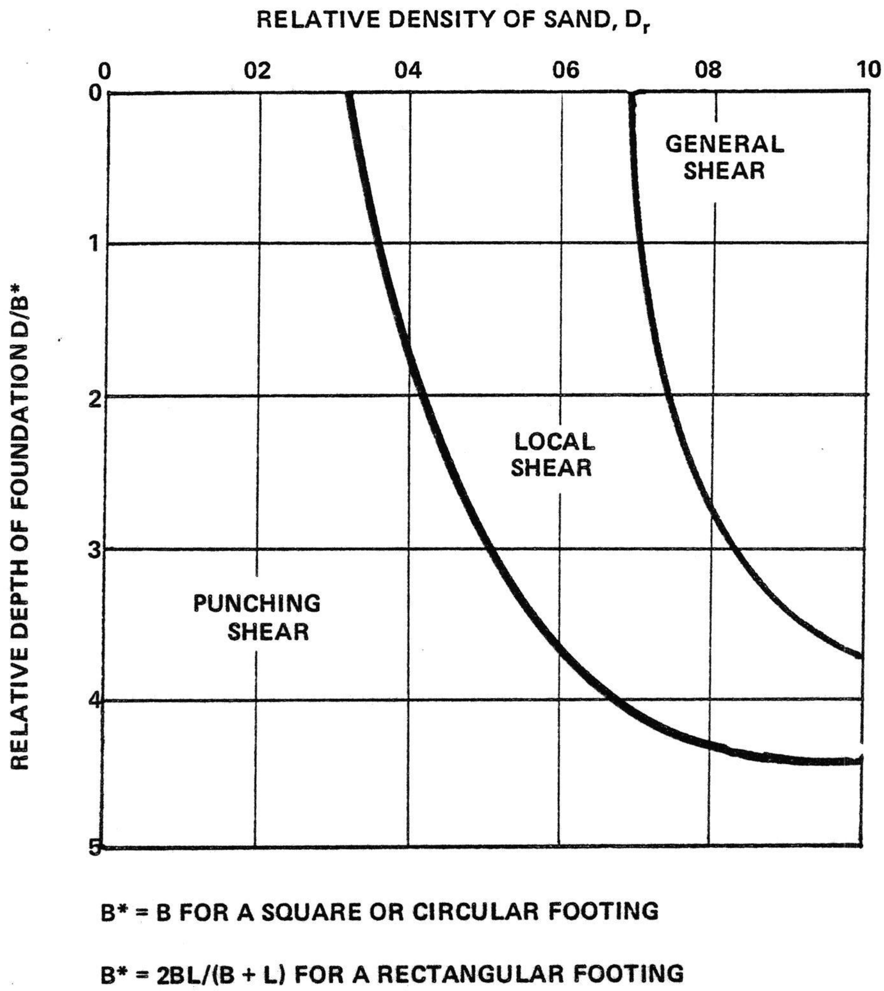
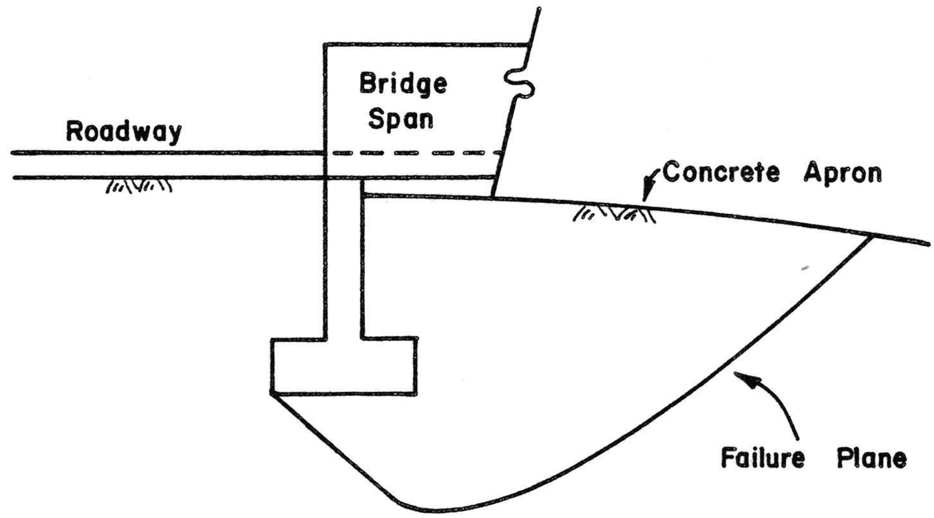


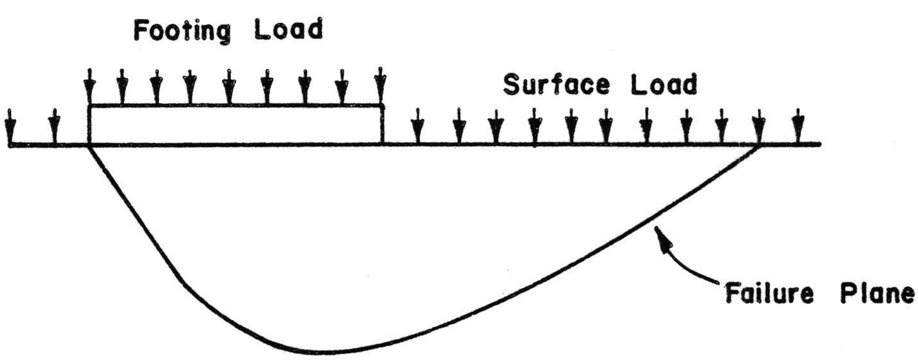
Figure 2-3. Modes of Failure of Model Footings in Chattahoochee Sand

Table 2-1. Bearing Capacity Factors

	N_c	N_q	N_γ (Vesic)	N_γ (Terzaghi)	N_γ (Meyerhof)
0	5.14	1.00	0.00	0.00	0.00
1	5.38	1.09	0.07	0.088	0.0022
2	5.63	1.20	0.15	0.182	0.0098
3	5.90	1.31	0.24	0.280	0.0228
4	6.19	1.43	0.34	0.384	0.0422
5	6.49	1.57	0.45	0.494	0.0700
6	6.81	1.72	0.57	0.622	0.106
7	7.16	1.88	0.71	0.761	0.152
8	7.53	2.06	0.86	0.912	0.210
9	7.92	2.25	1.03	1.07	0.279
10	8.35	2.47	1.22	1.25	0.366
11	8.80	2.71	1.44	1.46	0.471
12	9.28	2.97	1.69	1.70	0.595
13	9.81	3.26	1.97	1.96	0.743
14	10.37	3.59	2.29	2.23	0.922
15	10.98	3.94	2.65	2.54	1.13
16	11.63	4.34	3.06	2.94	1.38
17	12.34	4.77	3.53	3.38	1.66
18	13.10	5.26	4.07	3.87	2.00
19	13.93	5.80	4.68	4.40	2.40
20	14.83	6.40	5.39	4.97	2.87
21	15.82	7.07	6.20	5.75	3.42
22	16.88	7.82	7.13	6.61	4.07
23	18.05	8.66	8.20	7.55	4.82
24	19.32	9.60	9.44	8.58	5.71
25	20.72	10.66	10.88	9.70	6.76
26	22.25	11.85	12.54	11.35	8.00
27	23.94	13.20	14.47	13.16	9.46
28	25.80	14.72	16.72	15.15	11.19
29	27.86	16.44	19.34	17.33	13.23
30	30.14	18.40	22.40	19.73	15.67
31	32.67	20.63	25.99	22.80	18.56
32	35.49	23.18	30.22	26.62	22.03
33	38.64	26.09	35.19	31.07	26.16
34	42.16	29.44	41.06	36.46	31.15
35	46.12	33.30	48.03	42.43	37.16
36	50.59	37.75	56.31	50.52	44.42
37	55.63	42.97	66.19	58.7	53.27
38	61.35	48.93	78.03	70.1	64.07
39	67.87	55.96	92.25	80.0	77.34
40	75.31	64.20	109.41	100.4	93.70
41	83.86	73.90	130.22	121.7	113.99
42	93.71	85.38	155.55	149.5	137.68
43	105.11	99.02	186.54	185.2	171.15
44	118.37	115.31	224.64	232.8	211.41
45	133.88	134.88	271.76	297.5	262.75
46	152.10	158.51	330.35	381.5	328.7
47	173.64	187.21	403.67	500.9	414.3
48	199.26	222.31	496.01	656.8	526.5
49	229.93	265.51	613.16	868.1	674.9
50	266.89	319.07	762.89	1,153.2	873.9



Typical Bridge Footing



Simplified Footing

Figure 2-4.

Solution:

Since the footing is at the surface, there is no surcharge load q. From Table 2-1,

$$N_c = 14.83$$
$$N_q = 6.40$$
$$N_\gamma = 5.39$$

and

$$q_o = \frac{100(3)}{2}(5.39) + 300\,(14.83) + 0\,(6.40)$$
$$q_o = \underline{\underline{5{,}258 \text{ psf}}}$$

Example 2:

If the footing of example 1 had been placed at a depth of 4 ft., what would be the ultimate bearing capacity?

Solution:

The surcharge load is now 400 psf (4 ft. X 100 pcf).

$$q_o = \frac{100(3)}{2}(5.39) + 300\,(14.83) + 400\,(6.40)$$
$$q_o = \underline{\underline{7{,}818 \text{ psf}}}$$

The allowable bearing capacity, q_{all}, is the ultimate bearing capacity divided by a factor of safety, usually 2.5 or 3.

$$q_{all} = \frac{q_o}{F.S.} \qquad \text{(Equation 2-2)}$$

Example 3:

What width is required to support a long surface footing with a load of 2,000 lb per ft? The soils c = 150 psf, $\phi = 25^o$, and γ = 110 pcf. Use the bearing capacity factors suggested by Vesic and a safety factor of 3.

Solution:

From Table 2-1,

$$N_c = 20.72$$
$$N_q = 10.66$$
$$N_\gamma = 10.88$$

The allowable bearing pressure is the load divided by the area.

$$q_{all} = \frac{2000\frac{LB}{FT}}{B\ FT} = \frac{q_o}{3}$$

or

$$\frac{6000}{B} \text{ psf} = q_o = \frac{110B}{2} (10.88) + 150 (20.72) + 0 (10.66)$$

$$B^2 + 5.19 B - 10.03 = 0$$

$$\underline{\underline{B = 1.5 \text{ FT}}}$$

EFFECT OF FOOTING SHAPE

Shapes other than a strip footing are solved on a semi-empirical basis. Exact solutions exist only for circular shapes and these have not been proved by field observation (Hansen and Christensen, 1969).

Ultimate bearing capacity for other than strip footings can be found by multiplying terms of Equation 2-1 by shape factors.

$$q_o = \frac{\gamma B}{2} N_\gamma \cdot s_\gamma + c N_c s_c + {}_q N_q s_q \qquad \text{(Equation 2-3)}$$

where:

s_γ = shape factor applied to N_γ term
s_c = shape factor applied to N_c term
s_q = shape factor applied to N_q term

Values for shape factors are found in Table 2-2. Shape factors in Table 2-2 were developed by DeBeer (1970) and modified by Vesic (1975).

Table 2-2: Shape Factors for Shallow Foundations. (After DeBeer, 1967, as modified by Vesic, 1975).

$$s_c = 1 + (\frac{B}{L}) (N_q/N_c)$$

$$s_q = 1 + \frac{B}{L} \tan \phi$$

$$s_\gamma = 1 - 0.4 \frac{B}{L}$$

Example 4:

Find the ultimate bearing capacity of a 3 foot square footing that is 4 feet deep. Soil properties are:

c = 300 psf
ϕ = 20°
γ = 100 pcf

Solution:

From Table 2-1

N_c = 14.83 N_q/N_c = .43

N_q = 6.40

N_γ = 5 .39 $\tan \phi$ = .36

and according to Table 2-2

s_γ = .60

s_c = 1 + .43

s_q = 1 + .36

From Equation 2-3

$$q_o = \frac{100\ (3)}{2}\ (5.39)\ (.60) + 300\ (14.83)(1.43) + 400\ (6.40)\ (1.36)$$

$$q_o = 10{,}329 \text{ psf}$$

EFFECT OF ECCENTRIC LOAD

Eccentric loads are caused by non-concentric placement of loads on footings or by moments transmitted to the footings (Figure 2-5). An eccentric load shifts the location of the general shear failure wedge toward the load causing the bearing capacity to be reduced. A smaller failure plane is developed and stress is reduced on the side away from the load.

Bearing capacity can safely be determined by reducing the width by twice the eccentricity (Figure 2-5) (Meyerhof, 1953).

$$B' = B - 2e \qquad \text{(Equation 2-4a)}$$

In order to prevent uplift on the side away from eccentricity, the load should be kept in the center third (i.e. $e < 1/6\ B$) of the footing.

When a load is eccentric in two directions, i.e. in the longitudinal direction as well, the effective length must also be reduced.

$$L' = L - 2e_1 \qquad \text{(Equation 2-4b)}$$

The ultimate stress then computed is an average ultimate stress:

$$q_o = \frac{Q_o}{B'L'} \qquad \text{(Equation 2-5)}$$

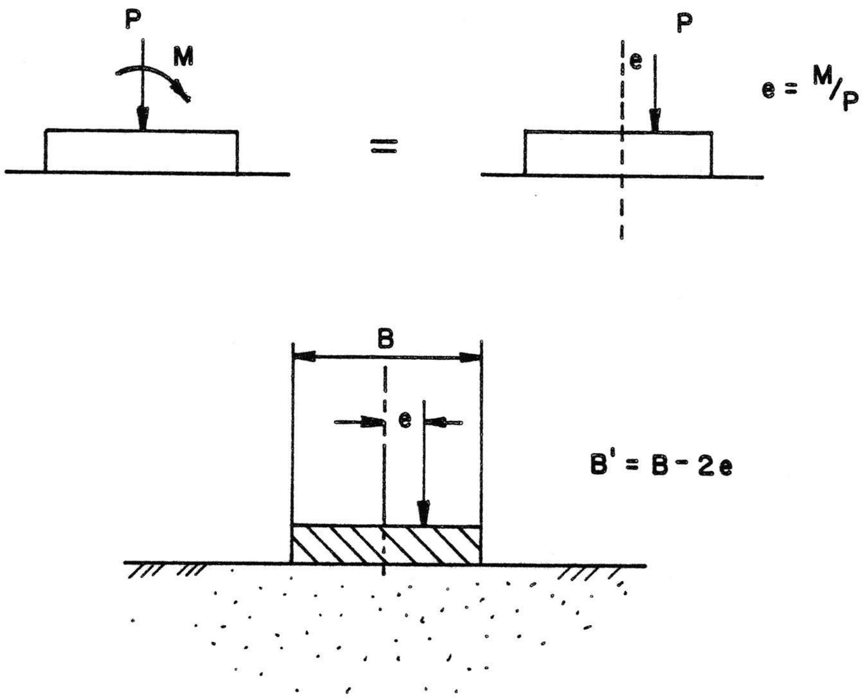
where

q_o = average ultimate stress

Q_o = ultimate load (force)

B' = effective width

L' = effective length



ECCENTRIC LOAD

Figure 2-5

In turn, the width B used on computing the ultimate stress, q_o. from equation 2-1 is the least of B' or L'. (L' may be less than B' if the eccentricity in the longest direction is larger than the eccentricity across its width.)

$$q_o = \frac{\gamma B'^*}{2} N_\gamma + \bar{c} N_c + qN_q = \frac{Q_o}{B'L'} \qquad \text{(Equation 2-6)}$$

*use the least of B' or L' in the N_γ term.

Example 5:

Find the ultimate load of a 6 ft. square footing two feet deep with the load offset .5 ft. on one side and 1.0 ft. on the other. Soil properties are:

$$c = 0$$
$$\phi = 30^o$$
$$\gamma = 100 \text{ pcf}$$

Solution:

From Table 2-1 (Factors by Vesic)

$$N_\gamma = 22.40$$
$$N_q = 18.40$$

From Table 2-2

$$s_\gamma = .60 \qquad \tan 30^o = .577$$
$$s_q = 1.577$$

and

$$B' = 6 - 2(.5) = 5' \qquad \text{(Equation 2-4a)}$$
$$L' = 6 - 2(1.0) = 4' \qquad \text{(Equation 2-4b)}$$

and applying equation 2-6 with shape factors

$$q_o = \frac{Q_o}{(5)\ (4)} = \frac{(100)\ (4)}{2} (22.4)\ (.60) + 0 + 2(100)\ (18.40)\ (1.577)$$

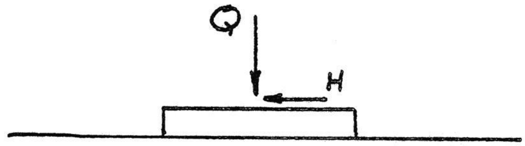
$$Q_o = 170{,}000 \text{ LB}$$

or with a safety factor of 2.5, the allowable load becomes

Qall = 68 KIPS

EFFECT OF INCLINED LOAD

An inclined load has a horizontal component, H, which tends to slide the footing and a vertical component, Q, which may cause a shear failure. Sliding seldom occurs at depth but H is limited at the surface by the shear strength of the soil:



$$H = Q \tan \phi + Ac_a \qquad \text{(Equation 2-7)}$$

where

- H = the maximum horizontal force
- Q = vertical component of load
- ϕ = angle of internal soil friction
- A = effective bearing area
- c_a = adhesion (equal to undrained shear strength in soft clays and negligible in sands)

In cases where sliding is not a problem, a reduction must be made in bearing capacity. Vesic (1973) suggests applying inclination factors i_γ, i_c, and i_q.

$$i_\gamma = \left\{1 - \frac{H}{Q + BLc \cot \phi}\right\}^{m+1} \qquad \text{(Equation 2-8a)}$$

where

$$m_B = \frac{2 + B/L}{1 + B/L} \qquad \text{(Equation 2-8b)}$$

is applied for m if the load is inclined to the width (shorter side) of the footing and

$$m_L = \frac{2 + L/B}{1 + L/B} \qquad \text{(Equation 2-8c)}$$

Equation 2-8c is applied when the load is inclined to the length (longer side) of the footing.

$$i_q = \left\{1 - \frac{H}{Q + BLc \cot\phi}\right\}^{m} \qquad \text{(Equation 2-8d)}$$

$$i_c = i_q - \frac{1 - i_q}{N_c \tan\phi} \qquad \text{(Equation 2-8e)}$$

In the special case where $\phi = 0$,

$$i_c = 1 - \frac{mH}{BLcN_c} \qquad \text{(Equation 2-8f)}$$

So the general equation for a rectangular footing with an inclined load becomes

$$q_o = \frac{\gamma B}{2} N_\gamma s_\gamma i_\gamma + c N_c s_c i_c + qN_q s_q i_q \qquad \text{(Equation 2-9)}$$

Example 6:

Find the ultimate load on a 6 ft. square footing two feet deep with the load inclined to one side at 10° from the vertical. Soil properties are

$$c = 0$$
$$\phi = 30^\circ$$
$$\gamma = 100 \text{ pcf}$$

From Table 2-1 and Table 2-2

$N_\gamma = 22.40$ $\quad s_\gamma = .60$

$N_q = 18.40$ $\quad s_q = 1.577$

From statics

$$H = Q \tan 10^\circ = .1760\ (Q)$$

From Equation 2-8b

$$m = \frac{2 + 1/1}{1 + 1/1} = 1.5$$

From Equation 2-8a

$$i_\gamma = \left\{ 1 - \frac{.176Q}{Q+0} \right\}^{1+1.5} = .616$$

From Equation 2-8d

$$i_q = \left\{ 1 - \frac{.176Q}{Q+0} \right\}^{1.5} = .748$$

Finally from Equation 2-9

$$q_o = \frac{Q}{(6)\ (6)} = \frac{100\ (6)}{2} (22.4)\ (.60)\ (.616) + 0 + 2(100)\ (18.40)\ (1.577)\ (.748)$$

$Q = 246^K$ Vertical Load

$H = 43^K$ Horizontal Load

EFFECT OF GROUND SURFACE SLOPE

Due to the necessity to span rivers and streams and place footings in approach embankments, bridge footings must account for ground surface slopes. Often the surface slopes down from the apron side of the footing (Figure 2-4) causing the bearing capacity to be reduced.

Solutions were developed by Hansen (1970) and refined by Vesic (1975) for the plane strain condition, i.e. a strip footing. The solutions are valid so long as the slope, ω, is less than the angle of internal friction or 45°.

$$\omega < \phi$$
$$\omega < 45^\circ$$

In addition, the solutions do not account for existing shearing stresses in the soil. Existing shearing stresses will likely be negligible if the slope is limited to half the angle of internal friction.

$$0 < \omega < \phi/2$$

Slopes greater than $\phi/2$ should be investigated for slope stability failure.

Correction factors, ω_γ ω_q, and ω_c, may be applied to the general bearing capacity equation as was done to correct for inclination of load to account for ground slope.

$$\omega_\gamma = \omega_q = \left\{1 - \tan\omega\right\}^2 \qquad \text{(Equation 2-10a)}$$

and

$$\omega_c = \omega_q - \left\{\frac{1 - \omega_q}{N_c \tan\phi}\right\} \qquad \text{(Equation 2-10b)}$$

In addition, the inclined slope distributes the surcharge load over a greater area causing the q term to become

$$q = \gamma d \cos\omega \qquad \text{(Equation 2-10c)}$$

Although the relations have not been proved experimentally, the application of shape factors and inclination factors are assumed to be valid. The general bearing capacity equation for a footing with ground slope on frictional soils ($\phi \neq 0$) then becomes:

$$q_o = \frac{\gamma B}{2} N_\gamma S_\gamma \omega_\gamma + cN_c S_c \omega_c + qN_q S_q \omega_q \qquad \text{(Equation 2-11a)}$$

Cohesive soils ($\phi = 0$) require the addition of the third weight term (Vesic, 1970) in the bearing capacity equation. As a result, N_γ becomes negative and equal to:

$$N_\gamma^* = -2\sin\omega \qquad \text{(Equation 2-10d)}$$

*N_γ from equation 2-10d is to be used only with soils where $\phi = 0$.

and equation 10 b reduces to

$$\omega_c = 1 - 2\omega/(\pi + 2) \qquad \text{(Equation 2-10e)}$$

where ω is in radians

The general bearing capacity equation for a footing with ground slope when $\phi = 0$ is (Vesic, 1976):

$$q_o = c\,N_c s_c \omega_c + \gamma D \cos\omega - \gamma B s_\gamma \sin\omega \qquad \text{(Equation 2-11b)}$$

Example 7:

Find the ultimate load of a 6 foot square footing 4 feet deep with ground slope of 1 vertical to 4 horizontal if the soil conditions are

	a)	b)
	$\gamma = 100$ pcf	$\gamma = 110$
	$c = 0$	$c = 1000$ psf
	$\phi = 30^o$	$\phi = 0$

Solution:

Soil condition a)

From Table 2-1		From Table 2-2	
N_γ =	22.40	s_γ =	0.60
N_c =	30.14	s_c =	1.61
N_q =	18.40	s_q =	1.58

$\omega = \tan^{-1}(1/4) = 14^o = .245$ rad.)

From Equation 2-10a

$$\omega_\gamma = \omega_q = (1 - .25)^2 = .56$$

From Equation 2-10b

$$\omega_c = .56 - \frac{1 - .56}{(30.14)(.577)} = .53$$

And from Equation 2-11a

$$\frac{Q}{(6)\ (6)} = \frac{100(6)}{2}(22.4)(.60)(.56) + 0 + 4(100)(.97)(18.4)(1.58)(.56)$$

Q = 309 kips

Soil Condition b)

From Table 2-1 & Equation 2-10d — From Table 2-2 — Equation 2-10e

N_γ = -.48	s_γ = .60	ω_c = .90
N_c = 5.14	s_c = 1.19	
N_q = 1.00	s_q = 1.00	

$$q_o = \frac{Q}{(6)(6)} = 1000\,(5.14)\,(1.19)\,(.90) + 110\,(4)\,(.97) - 110\,(6)\,(.60)\,(.24)$$

Q = 210 kips

EFFECT OF WATER TABLE

A water table that occurs within the zone of general shear failure (Figure 2-2a) will reduce the ultimate bearing capacity. Buoyancy reduces the weight of the soil and in turn the frictional forces within the soil. In clays, where friction develops little strength, the reduction in bearing capacity is negligible unless the cohesion is reduced as a result of a high water table over an extended period of time.

Because of strength reduction, the bearing capacity should be determined using the highest groundwater level expected during the life of the structure. In the case of bridges spanning rivers and streams, the water table should be assumed at the ground surface unless the designer is sure the water table will remain below that level.

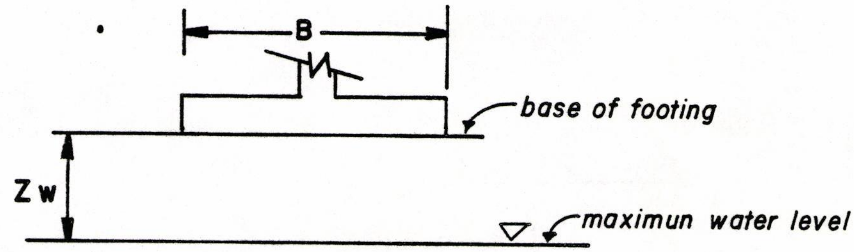
When the water table is assumed at or above the footing base, the buoyant weight, γ', i.e. the total saturated weight less the weight of water, should be used in the N_γ term of equation 2-1. If the water table is below a depth equal to the footing width B, the total unit weight γ can be used with the N_γ term. For a water table within a zone B below the footing, an average γ is used (Figure 2-6).

$$\gamma_{avg} = \frac{\gamma'\,(B - Zw) + \gamma\,Zw}{B} \qquad \text{(Equation 2-12)}$$

Reduction in q due to a water table above the footing base must also be made. In calculating the surcharge, q, for the N_q term of equation 2-1, grain to grain or effective stresses are used. The surcharge, q, then is the weight of all soil or surcharge above the footing, less the buoyant weight of water for the depth of water above the footing base.

Example 8:

A long strip footing, 4 feet wide and 3 feet deep, has a water table at the 5 foot depth. Find the ultimate bearing capacity if



For Z_w = 0 to B,

$$\gamma_{avg} = \frac{\gamma'(B - Z_w) + \gamma Z_w}{B}$$ *(equation 2-12)*

For Z_w greater than B

$$\gamma_{avg} = \gamma$$

Effect of Water Table on Unit Weight for N_γ Term

Figure 2-6

ϕ = 20°

c = 300 psf

γ = 100 pcf above the water table

γ = 112.4 pcf below the water table

Solution:

From Equation 2-12,

$$\gamma_{avg} = \frac{50(4-2) + 100\ (2)}{4} = 75 \text{ pcf}$$

and From Table 2-1,

N_γ = 5.39

N_c = 14.83

N_q = 6.40

Using Equation 2-1,

$$q_o = \frac{75\ (4)}{2}\ (5.39) + 300\ (14.83) + 3\ (100)\ (6.40)$$

$$q_o = 7.2 \text{ kips/ft.}^2$$

Example 9:

Find the ultimate bearing capacity of Example 8 if the water table rises 3 feet to a depth of 2 feet.

Solution:

γ_{avg} becomes γ' and the surcharge becomes

q = 2(100) + (112.4) - 1 (62.4) = 250 psf

and using Equation 2-1,

$$q_o = \frac{50(4)}{2}\ (5.39) + 300\ (14.83) + 250\ (6.40)$$

$$q_o = 6.9 \text{ kips/ft.}^2$$

EFFECT OF COMPRESSIBILITY

The type of bearing capacity failure which will occur, i.e. general shear, local shear or punching shear, depends on the size of the foundation and compressibility of the soil. The average shear strength mobilized along a failure plane below a footing decreases with increased foundation size (DeBeer, 1963 and 1965; Vesic 1964 and 1965; Kerisel, 1967).

According to Vesic (1973):

> "There are three independent reasons for this decrease of strength with foundation size: (1) curvature of Mohr envelope; (2) progressive rupture along the slip line; and (3) presence of zones or seams of weakness in all soil deposits. The relative contribution of each of the reasons varies with soil type and the range of footing size. Studies also show that the relative compressibility of soils both with respect to gravity forces and with respect to the soil strength, increases with the foundation size....It is postulated that the bearing capacity of large surface footings cannot be greater than the resistance of deep footings on the same soil. This postulate surmises that very large footings should fail exclusively in punching shear, as apparently all deep footings do. This should not be surprising, if the aforementioned fact that the relative compressibility of soils increases with footing size is considered."

In an attempt to quantify the effect of compressibility Vesic (1973) introduced compressibility factors, $\zeta_{\gamma c}$, ζ_{cc}, and ζ_{qc}, for each of the expressions in equation 1. Vesic (1975) stated:

> "The purpose of publishing these equations at this time is to allow the designer, in the absence of any other rational method, to assess numerically the order of magnitude of expected reduction of bearing capacity caused by the compressibility effects."

$$\zeta_{\gamma c} = \zeta_{qc} = \exp\{[(-4.4 + 0.6\ B/L) \tan \phi] + [(3.07 \sin \phi)(\log_{10} 2I_r)/(1 + \sin \phi)]\}$$

(Equation 2-13a)

$$\zeta_{cc} = \zeta_{qc} - \frac{1 - \zeta_{qc}}{N_c \tan \phi}$$

(Equation 2-13b)

for $\phi = 0$ $\quad \zeta_{cc} = 0.32 + 0.12\ B/L + 0.60 \log_{10} I_r$ (Equation 2-13c)

where

$$I_r = \frac{G}{c + q \tan\phi}$$

(Equation 2-13d)

in which q is the average normal stress at the depth of B/2 below the footing

G is the shear modulus

$$G = \frac{E}{2(1+\nu)}$$

(Equation 2-13e)

E = Modulus of Elasticity (Table 2-3)

ν = Poissons Ratio (Table 2-4)

Poissons ratio may also be taken as

$$\nu = \frac{K_o}{1 + K_o}$$

(Equation 2-13f)

where $K_o = 1 - \sin 1.2\phi$ (Equation 2-13g)

Table 2-3. Typical Range of Values for Modulus of Elasticity (E). (From Kezdi, 1975)

Type of Soil	Modulus of Elasticity psi	kp/cm^2
Very soft clay	50-400	3.5-30
Soft clay	250-600	20-50
Medium clay	600-1200	40-80
Hard clay	1,000-2,500	70-180
Sandy clay	4,000-6,000	300-400
Silty sand	1,000-3,000	70-200
Loose sand	1,500-3,500	100-250
Dense sand	7,000-12,000	500-800
Dense sand and gravel	14,000-28,000	1,000-2,000

Table 2-4. Range of Poisson's Ratio (ν) (From Barkan (1962) and others)

Soil Type	Poisson's Ratio
Clay, saturated	0.50
Clay with sand and silt	0.30-0.42
Clay, unsaturated	0.35-0.40
Loess	0.44
Sandy soil	0.15-0.25
Sand	0.30-0.35

Values of compressibility factors from equations 2-13a, 2-13b, and 2-13c are limited to 1.0 because a value of unity indicates a general shear failure. A check to see if compressibility factors should be applied can be made by comparing the rigidity index (Equation 2-13c) with the critical rigidity index (Equation 2-14)

$$(I_r)_{crit} = \tfrac{1}{2} \exp\left[(3.30 - 0.45\, B/L) \cot(45 - \phi/2)\right] \qquad \text{(Equation 2-14)}$$

A rigidity index smaller than the critical rigidity index requires a reduction in bearing capacity by applying the compressibility factors (equations 2-13a and 2-13b). Values of $(I_r)_{crit}$ for strip or square footings are given in Table 2-5.

TABLE 2-5. VALUES OF CRITICAL RIGIDITY INDEX.

Angle of Shearing Resistance ϕ	Critical Rigidity Index for: Strip Foundation B/L = 0	Square Foundation B/L = 1
0	13	8
5	18	11
10	25	15
15	37	20
20	55	30
25	89	44
30	152	70
35	283	120
40	592	225
45	1442	486
50	4330	1258

After Vesic, 1973

Values of compressibility factors ζ_{qc} and $\zeta_{\gamma c}$, for strip or square footings are given in Table 2-6 and shown in Figure 2-7. Table 2-7 gives the values for ζ_{cc} for strip and square footings.

Example 10:

A 10' X 30' rectangular footing exists at the 15 ft. depth. The subsoil is a sand with the following properties

E = 220 TSF (from Triaxial Test)

$\phi = 35^o$

c = 0

γ = 120 pcf

Find the ultimate bearing capacity

Table 2-6. Values of Compressibility Factor ζ_{qc}.

B/L = 1 (Square)

ϕ \ I_r	*1*	*2.5*	*5*	*10*	*25*	*50*	*100*	*250*	*500*
0°	1.000	1.000	1.000	(1.039)	·	·	·	·	·
5°	0.772	0.852	0.917	0.988	(1.090)	·	·	·	·
10°	0.587	0.703	0.806	0.924	(1.107)	·	·	·	·
15°	0.437	0.562	0.679	0.821	(1.056)	·	·	·	·
20°	0.317	0.433	0.548	0.694	0.948	(1.199)	·	·	·
25°	0.224	0.322	0.423	0.557	0.801	(1.054)	·	·	·
30°	0.152	0.228	0.310	0.422	0.634	0.863	(1.175)	·	·
35°	0.098	0.153	0.214	0.300	0.468	0.655	0.918	(1.433)	·
40°	0.059	0.096	0.137	0.197	0.317	0.456	0.654	(1.055)	·
45°	0.033	0.054	0.080	0.117	0.194	0.284	0.417	0.692	(1.015)
50°	0.016	0.027	0.041	0.061	0.104	0.155	0.231	0.393	0.587

B/L = 0 (Strip)

ϕ \ I_r	*1*	*2.5*	*5*	*10*	*25*	*50*	*100*	*250*	*500*
0°	1.000	1.000	1.000	1.000	·	·	·	·	·
5°	0.733	0.808	0.870	0.937	(1.034)	·	·	·	·
10°	0.528	0.632	0.725	0.831	0.996	(1.142)	·	·	·
15°	0.372	0.478	0.578	0.699	0.899	(1.087)	·	·	·
20°	0.255	0.348	0.441	0.558	0.762	0.964	(1.220)	·	·
25°	0.169	0.243	0.320	0.421	0.605	0.796	(1.048)	·	·
30°	0.107	0.161	0.219	0.299	0.449	0.610	0.831	(1.248)	·
35°	0.064	0.100	0.141	0.197	0.307	0.431	0.603	0.941	(1.318)
40°	0.036	0.058	0.083	0.119	0.192	0.275	0.395	0.638	0.916
45°	0.018	0.030	0.044	0.064	0.107	0.156	0.229	0.380	0.557
50°	0.008	0.013	0.020	0.030	0.051	0.076	0.113	0.192	0.287

In area marked by dots take $\zeta_{qc} = 1$.

Table 2-7. Values of Compressibility Factor ζ_{cc} for $\phi = 0$.

B/L \ I_r	*1*	*2.5*	*5*	*10*	*25*	*50*	*100*	*250*
1	0.440	0.679	0.859	(1.039)	·	·	·	·
0	0.320	0.559	0.739	0.919	(1.157)	·	·	·

In area marked by dots take $\zeta_{cc} = 1$.

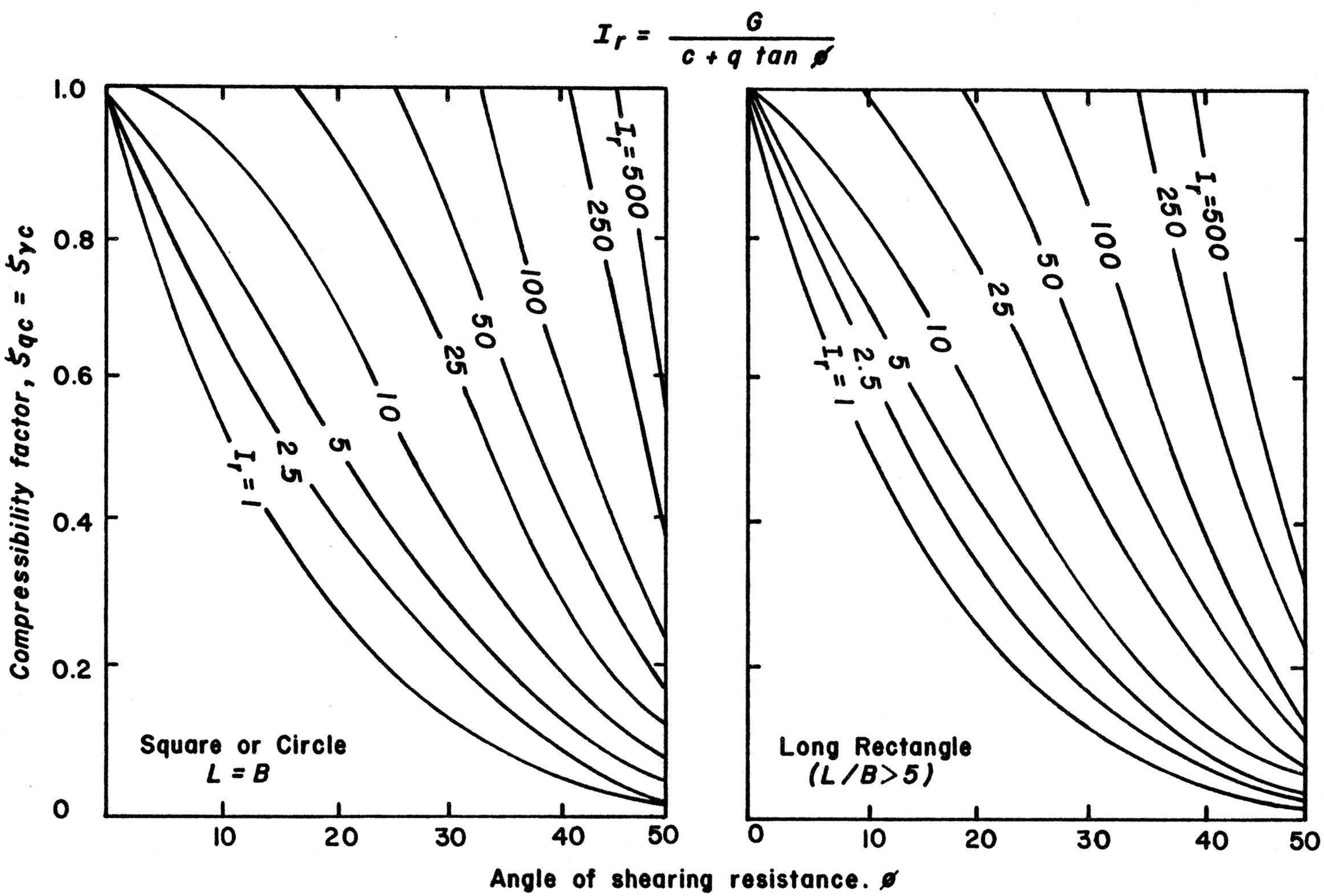


Figure 2-7.

Theoretical compressibility factors. (After Vesić, 1973)

Solution:

At a depth of B/2 in the elastic zone below the footing, the horizontal coefficient of earth pressure is (Equation 2-13g):

$$K_o = 1 - \sin (1.2)(35^o) = .33$$

and Poisson's ratio is (Equation 2-13f):

$$\nu = \frac{K_o}{1+K_o} = \frac{.33}{1+.33} = .25$$

and Shear Modulus is (Equation 2-13e):

$$G = \frac{E}{2(1+v)} = \frac{220\ TSF}{2(1+.25)} = 88\ TSF$$

The mean normal stress at B/2

$$q = \frac{20(120)}{2000} = 1.20\ TSF$$

Then according to Equation 2-13d

$$I_r = \frac{88}{0+1.20(.70)} = 105$$

and I_r critical (Equation 2-14):

$$I_{r\ crit} = ½ \exp \{ [3.30 - .45 (1/3)] \cot (45^o - \frac{35^0}{2}) \} = 212$$

$I_r < I_{rcrit}$ therefore the assumption of incompressibility is not justified and correction factors must be applied. From Equation 13a:

$$\zeta_{qc} = \zeta_{\gamma c} = \exp \{ [-4.4 + .6 (1/3)](.70) + \frac{3.07 (.57) (\log_{10} 2(105))}{(1+.574)} \} = .71$$

From Table 2-1

N_γ = 48.03

N_q = 33.30

From Table 2-2

S_γ = .87

S_q = 1.23

and from Equation 2-3 with compressibility factors

$$q_o = \frac{120(10)}{2} (48.03) (.87) (.71) + 15(120) (33.30) (1.23) (.71)$$

q_o = 35 TSF

SETTLEMENT

Both the amount and rate of settlement for shallow foundations can be calculated from the one dimensional consolidation test. Details of the theory and laboratory procedure for this test are contained in Chapter 5 of the 1975 interim report on Laboratory Investigations and in Chapter 10 of the 1977 interim report on Laboratory Procedures.

Details of how the amount and rate of settlement are calculated are included in example 11 below. Data for the example are developed in the examples of Chapter 10 from the 1977 report on Laboratory Procedures.

Example 11:

A footing 16 feet square overlies a clay layer 4 feet thick with two way drainage (Figure 2-8). The existing overburden pressure is 0.60 tons per square foot and the stress at the base of the footing is 3.43 tons per square foot. Figure 2-9 represents the laboratory results from the clay layer. Find the amount and rate of settlement due to primary consolidation under the center of the footing.

Solution:

The amount of settlement depends on the final pressure which is applied to the center of the clay layer. The final pressure is the sum of the 0.60 tons overburden pressure and the stress applied to the clay layer by the footing. Stress applied by the footing is reduced as the distance between the footing and clay layer is increased. In this example, the center of the clay layer is 8 feet below the footing, half the footing width. The stress applied by the footing is found from a Boussinesq stress distribution analysis (Figure 2-10). At half the footing width, under the center of a square footing, the stress is .7q or 70% of the original 3.43 tsf applied by the footing. The stress applied to the clay layer by the footing then becomes:

$$.7(3.43 \text{ tsf}) = 2.40 \text{ tsf}$$

The final pressure is:

$$0.60 \text{ tsf} + 2.40 \text{ tsf} = 3.00 \text{ tsf}$$

To find the total settlement, S, of the soil stratum due to 100% primary consolidation, equation 2-15 is used.

$$S = H \frac{e_1 - e_2}{1 + e_1} \qquad \text{(Equation 2-15)}$$

H = thickness of layer

e_1 = void ratio before load is applied

e_2 = final void ratio after consolidation under increased stress

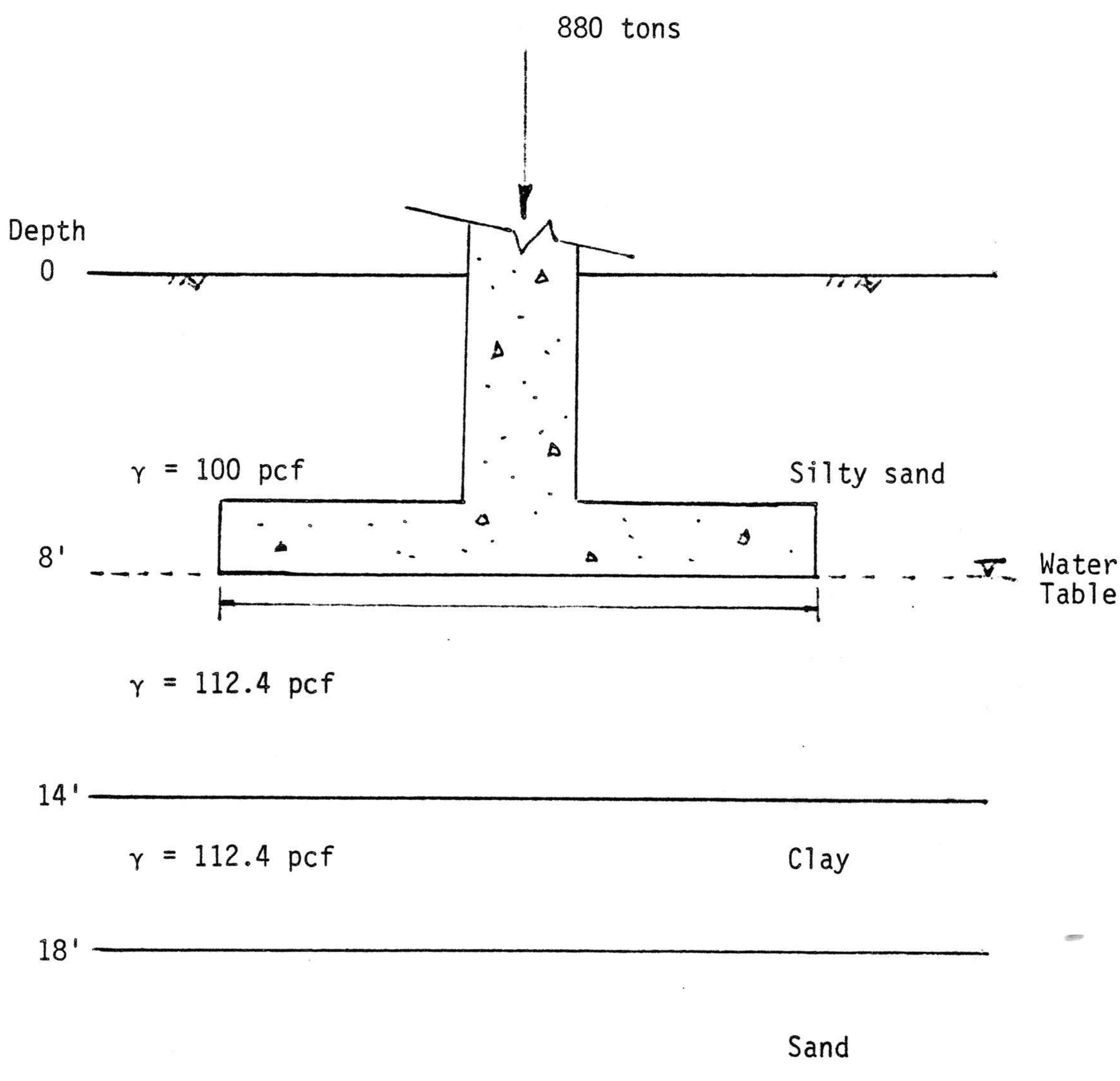


Figure 2-8. Soil Profile of Example 11.

CONSOLIDATION TEST
(Results)

Job No. 16677
Location L.R W.T.P.
Boring No. 5A
Sample No. 2
Depth 16ft

Test Date 23AUG76
Tested By LHB

Effective Overburden Pressure 0.600 tsf

Initial Water Content (w_a) 25.6%
Initial Void Ratio (e_i) 0.765
Initial Saturation (S_0) 90.4%

Preconsolidation Pressure (p_0') 2.0 tsf

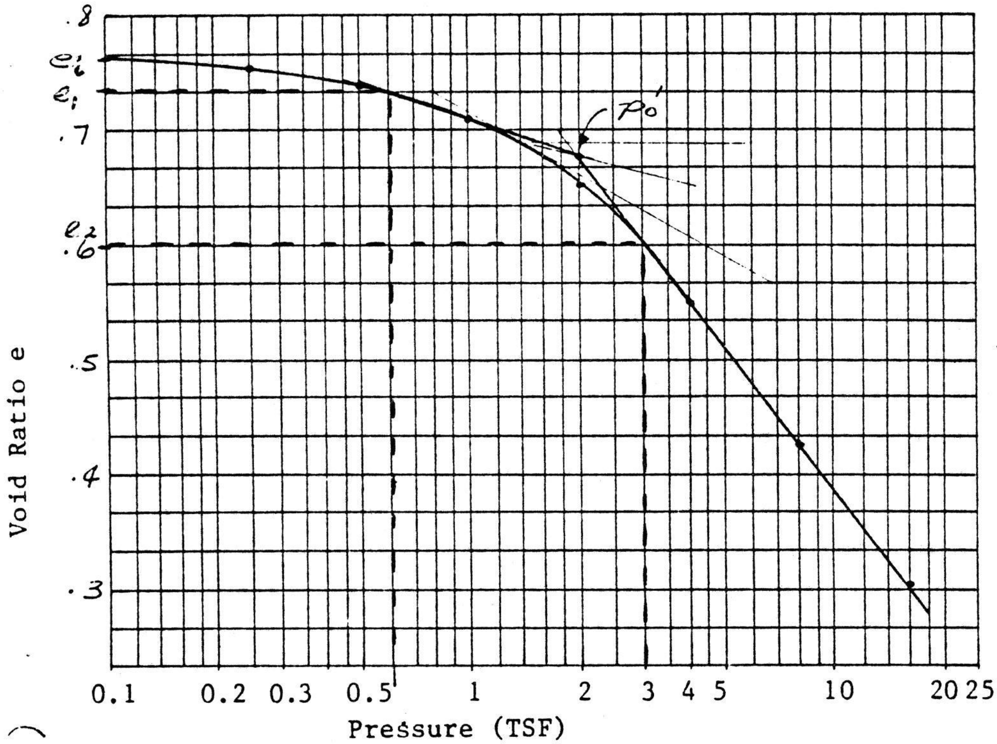
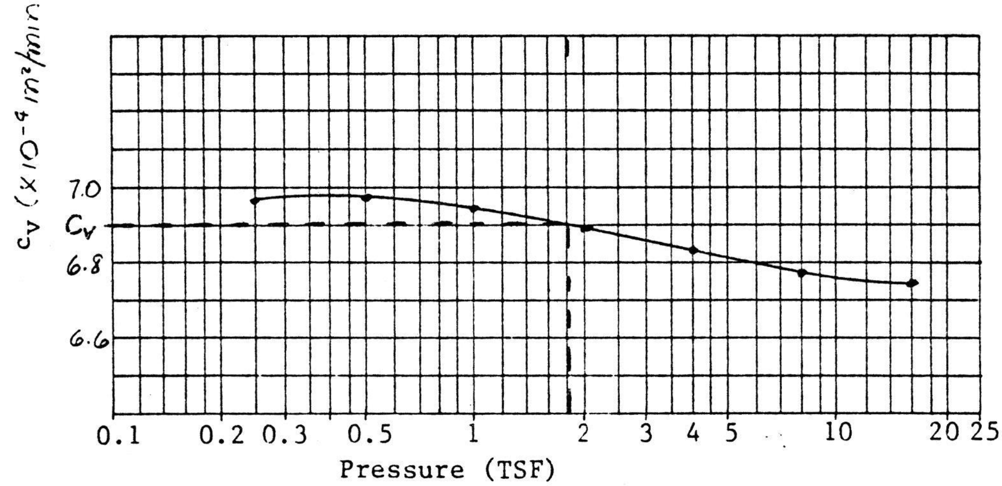


Figure 2-9



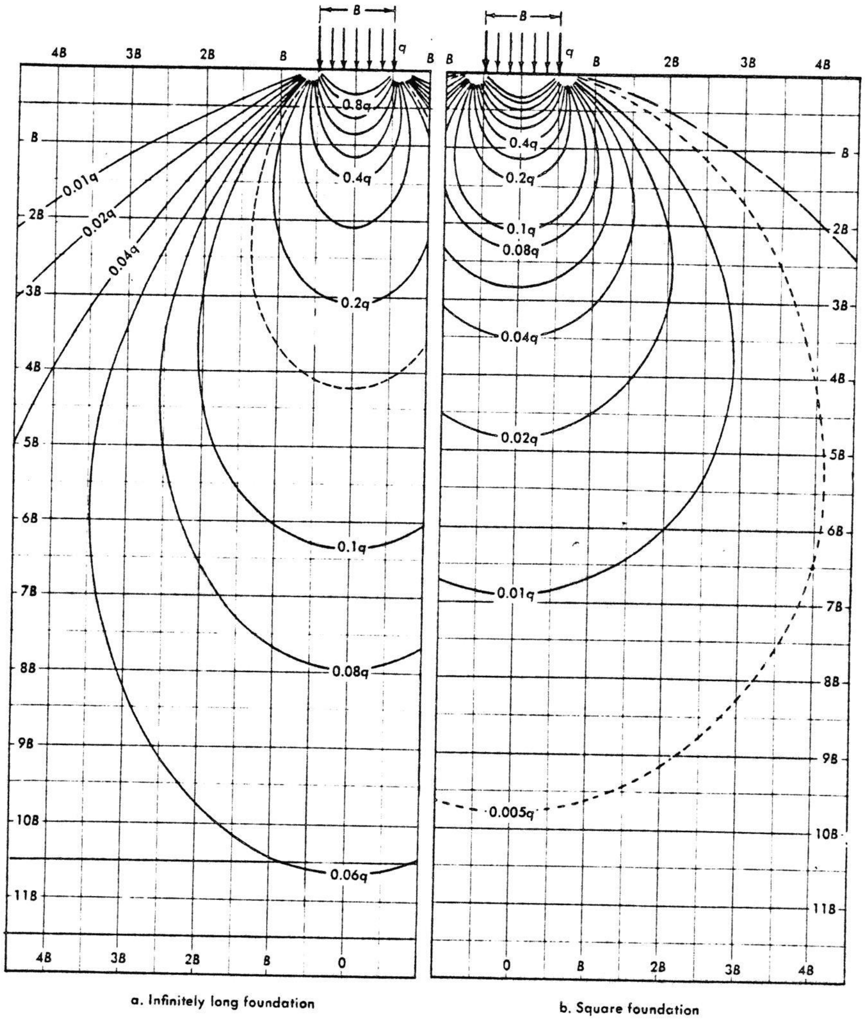


Figure 2-10. *Contours of equal vertical stress beneath a foundation in a semi-infinite elastic solid — the Boussinesq analysis. Stresses given as functions of the uniform foundation pressure q; distances and depths given as functions of the footing width B.*

From Sowers & Sowers, 1970.

The void ratio, e_1, which corresponds to the overburden pressure of 0.60 tsf is 0.733 (Figure 2-9). The void ratio, e_2, which corresponds to the final pressure of 3.00 tsf is 0.600.

Solving for S:

$$S = (48 \text{ inches}) \frac{(1.733 - .600)}{(1 + .733)}$$

S = 3.7 inches total settlement

The time, t, required to reach any degree of primary consolidation, U, is found from equation 2-16:

$$t = T \frac{\left[\frac{H}{N}\right]^2}{c_v} \qquad \text{(Equation 2-16)}$$

T = theoretical time factor for the specified degree of consolidation (Table 2-8).

H = thickness of consolidating layer

N = number of drainage faces for the consolidating layer

c_v = coefficient of consolidation

TABLE 2-8

Theoretical Time Factors

% Consolidation	U O 10	20	30	40	50	60	70	80	90
Time Factor	T 0.008	.031	.071	.126	.197	.287	.403	.567	.848

For the example, the thickness of the consolidating layer, H, is 48 inches; the number of drainage faces, N, is two; and the coefficient of consolidation, c_v, which corresponds to the average consolidating stress of 1.80 tsf is 6.9×10^{-4} in^2 /min (Figure 2-8). Solving for the time, t, in terms of the time factor T:

$$t = T \frac{\left[\frac{48 \text{ in.}}{2}\right]^2}{6.9 \times 10^{-4} in^2/min} \cdot \frac{1 \text{ day}}{1440 \text{ min}}$$

t = 580 T days

Applying the time factors, T, from Table 2-8, the rate of settlement is obtained for 10%, 20%. . .90% primary consolidation.

% Consolidation	10	20	30	40	50	60	70	80	90
Time in days	5	18	41	73	114	116	234	329	492

The amount of settlement in inches which corresponds to 10%, 20%. . .90% consolidation is 10%, 20%. . .90% of the total settlement of 3.7 inches.

% Consolidation	10	20	30	40	50	60	70	80	90
Settlement in inches	.37	.74	1.11	1.48	1.85	2.22	2.59	2.96	3.33

CHAPTER III

DEEP FOUNDATIONS

INTRODUCTION

Deep foundations for bridges will usually fall into one of the following categories:

1. Piles
2. Drilled shafts
3. Caissons

The design and analysis of all three types of foundations are basically similar. All deep foundations derive their capacity to support loads from a combination of friction or adhesion on the sides of the foundation element and bearing on the tip or base of the element. The relative values of skin friction and end bearing depend not only on the soil stratification but also to a large extent on construction procedures. For example, the driving of displacement piles usually increases the lateral earth pressure above the naturally existing value, while the excavation of drilled shafts will allow stress relief. On the other hand, the disturbance caused by displacement piles in clays generally reduces the available shear strength while the disturbance caused by the excavation of drilled shafts is relatively slight. Design of deep foundations is usually based on (1) soil properties inferred from field and/or laboratory tests, (2) field load tests of prototype foundations or elements, or (3) resistance to driving (for piles). All three procedures are commonly used (not necessarily simultaneously) for pile foundations. The design of drilled shaft foundations and caissons is usually based on measured soil properties, but occasionally load tests are performed on drilled shafts. The design of caissons is often modified as construction progresses.

DESIGN METHODS BASED ON SOIL PROPERTIES

Several methods are available for analysis and design of deep foundations based on measured or inferred soil properties. These are:

1. Empirical correlation with field tests
2. Limit equilibrium analysis
3. Load transfer function method
4. Elastic solid analysis
5. Finite element method

The methods are listed in order of increasing complexity. All of the methods except the

empirical correlation and the limit equilibrium analysis require the use of a computer for efficient application. As the sophistication of the method of analysis increases so does the need for sophisticated testing and accurate soil data. The engineer must determine the optimum method for each job, based on the job requirement, soil data, and computational facilities available to him.

EMPIRICAL CORRELATION WITH FIELD TESTS

A large amount of empirical data has been accumulated over the many years of use of the standard split-spoon penetrometer and quasi-static cone penetrometer relating to the correlation of pile capacity with penetration resistance. The summary of these correlations as given by Meyerhof (1976) is described in this section.

The ultimate capacity of deep foundations, Q_{ult}, may be expressed as the sum of frictional resistance, Q_{SF}, and end bearing resistance, Q_{EB}, or

$$Q_{ult} = Q_{SF} + Q_{EB} \tag{3.1}$$

The resistance due to skin friction may be expressed as

$$Q_{SF} = f_s A_s \tag{3.2}$$

where

f_s = average unit skin friction

A_s = surface area of pile or shaft acted upon by f_s

The end bearing resistance may be expressed as

$$Q_{EB} = q_{ult} A_t \tag{3.3}$$

where

q_{ult} = ultimate unit bearing capacity at pile or shaft tip

A_t = area of pile or shaft tip

Standard Penetration Test. For driven piles, the ultimate unit bearing capacity in tons per square foot is approximately

$$q_{ult} = \frac{0.4\,N\,D_b}{B} \leq 4N \tag{3.4}$$

N = average standard penetration resistance near the pile tip, corrected to an effective overburden pressure of 1 tsf. (See Figure 3.1).

D_b = depth to pile tip

B = pile width or diameter

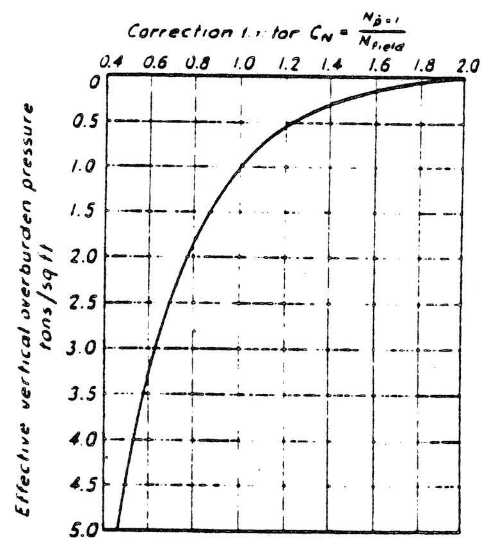


Figure 3.1. Chart for Correction of N-Values in Sand for Influence of Overburden Pressure (reference value of effective overburden pressure 1 ton/sq ft).

The limiting value of 4N given in Eq. 3.4 is recommended for sands and gravels but a limiting value of 3N is suggested for non-plastic silts. The empirical relation given by Meyerhof (1976) is shown in Figure 3.2. The ultimate skin friction of driven displacement piles in tons per square foot is roughly given by

$$f_s = \frac{\bar{N}}{50} \tag{3.5}$$

where

$\bar{N}$ = average standard penetration resistance for the embedded length of the pile. Meyerhof suggests an increase of 50 percent in skin friction values for driven piles with a taper exceeding about 1 percent. The empirical correlation between skin friction and penetration resistance is shown in Figure 3.3.

Quasi-static Cone Penetration Test. The cone penetrometer, equipped with a friction sleeve, develops approximately the same end bearing and friction values as a full size pile. Several empirical corrections are often required and reference should be made to Welch and Thornton (1978) or Schmertmann (1975) for details of the design procedure.

LIMIT EQUILIBRIUM ANALYSIS

In limit equilibrium analysis, a rigid-plastic deformation condition is assumed. The pile

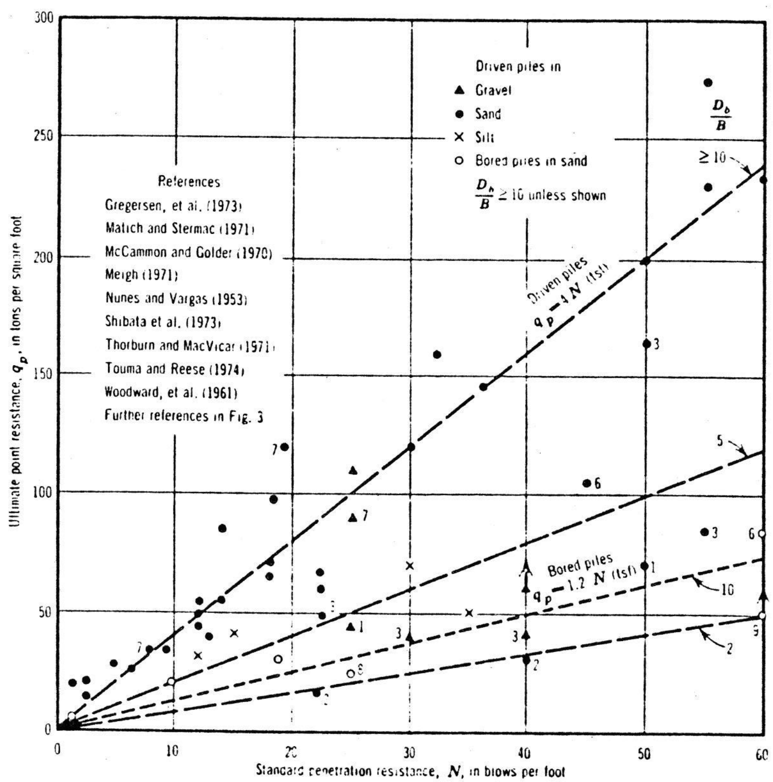


Figure 3.2. Empirical Relation between Ultimate Point Resistance of Piles and Standard Penetration Resistance in Cohesionless Soil.

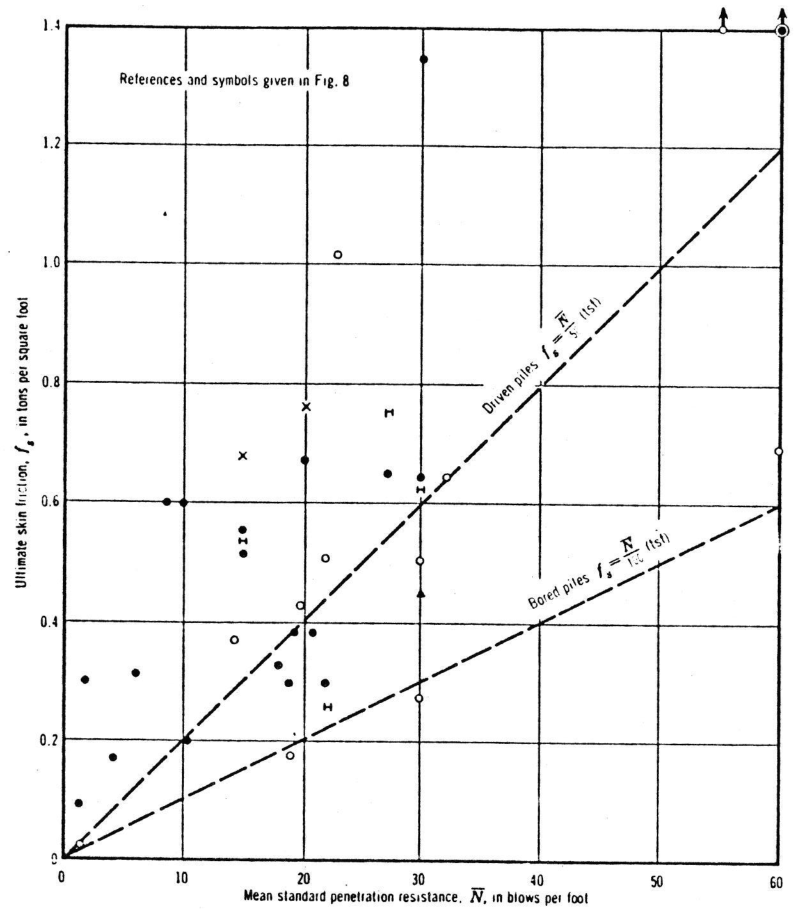


Figure 3.3. Empirical Relation between Ultimate Skin Friction of Piles and Standard Penetration Resistance in Cohesionless Soil.

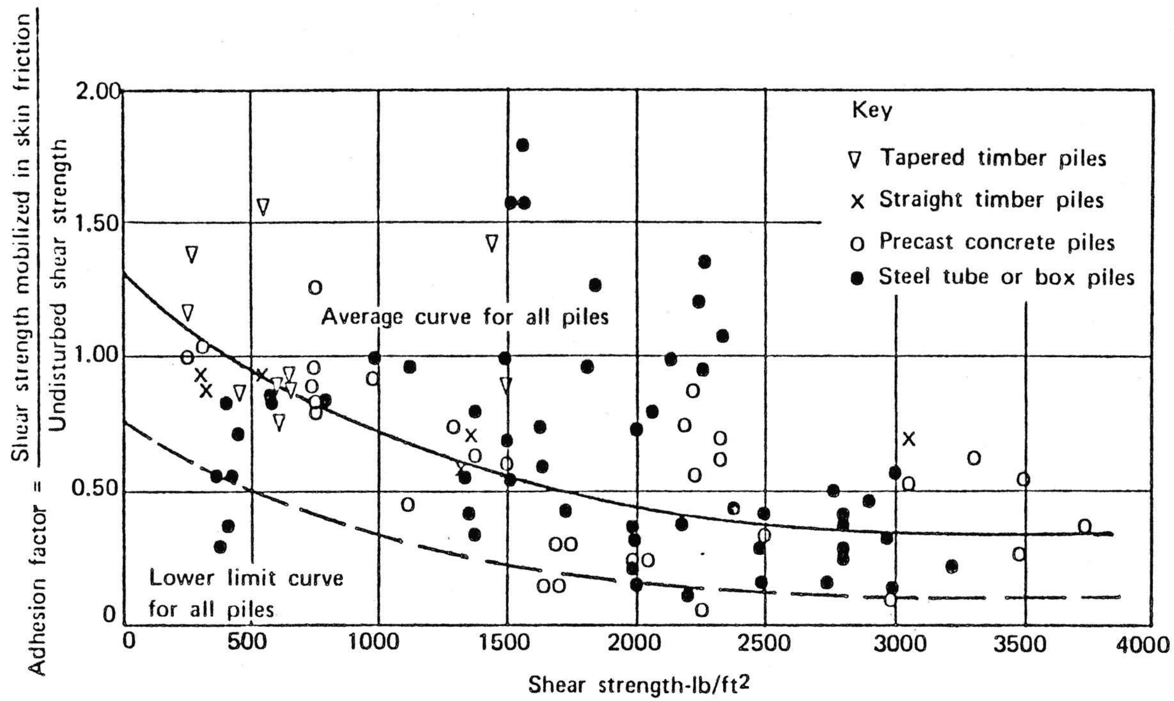


Figure 3.4. The Adhesion Factor as a Function of Shear Strength (after Tomlinson)

is considered incompressible and skin friction and end bearing reach their maximum values simultaneously. It is also assumed that loads transferred to the soil through friction or bearing do not influence the existing lateral or vertical earth pressures.

The ultimate capacity of a pile, Q_{ult}, can be determined by summing the total frictional resistance, Q_{SF}, and the maximum end bearing resistance, Q_{EB}.

$$Q_{ult} = Q_{SF} + Q_{EB} \tag{3.6}$$

The frictional resistance is the average friction or adhesion multiplied by the surface area of the pile.

$$Q_{SF} = f_{avg} PL \tag{3.7}$$

where:

f_{avg} = average unit skin friction or adhesion

P = perimeter of the pile

L = embedded length of the pile

The adhesion developed in clays is usually less than the shear strength or cohesion. Tomlinson (1969) has examined the relationship between skin friction in clays and the undisturbed shear strength. The ratio of skin friction to undisturbed shear strength is called the adhesion factor, α. A plot of α as a function of shear strength is shown in Figure 3.4. The skin friction of piles in clay can be determined by using Figure 3.4 and the following expression.

$$f = c\,\alpha \tag{3.8}$$

where:

c = undisturbed shear strength or cohesion

α = adhesion factor

The frictional resistance in sands is dependent upon the effective lateral earth pressure acting upon the pile surface and the coefficient of friction between the soil and the pile material. Above some critical depth, z_c, both vertical and horizontal effective stresses increase linearly with depth, but are essentially constant below the critical depth (Vesic, 1967). This critical depth is a function of relative density, D_r, and has been observed as follows:

$$\text{For } D_r < 30\%,\ z_c = 10D \tag{3.9}$$

$$\text{For } D_r > 70\%,\ z_c = 20D \tag{3.10}$$

where:

z_c = critical depth

D = pile diameter or width

The effective vertical stress in the vicinity of the pile can be determined as follows:

$$\text{For } z < z_c,\ \bar{p}_v = \bar{\gamma}\ z \tag{3.11}$$

$$\text{For } z \geq z_c,\ \bar{p}_v = \bar{\gamma}\ z_c \tag{3.12}$$

where:

$\bar{p}_v$ = effective vertical stress

$\bar{\gamma}$ = effective soil unit weight

z = depth below ground surface

The effective horizontal stress may be expressed as a function of the effective vertical stress.

$$\bar{p}_h = K_s \bar{p}_v \tag{3.13}$$

where:

$\bar{p}_h$ = effective horizontal stress

K_s = lateral pressure coefficient

The construction procedure has a significant influence on the lateral earth pressure and K_s. Values of K_s for various installation procedures (Sowers and Sowers, 1970) are given in Table 3.1.

TABLE 3.1
LATERAL EARTH PRESSURE
COEFFICIENT IN COHESIONLESS SOILS

Soil	Displacement Condition	K_s
Loose sand	Jetted Pile	0.5 to 0.75
(Dr 30%)	Drilled Pile	0.75 to 1.5
	Driven Pile	2 to 3
Dense sand	Jetted Pile	0.5 to 1
(D_r 70%)	Drilled Pile	1 to 2
	Driven Pile	3 to 5

The frictional resistance of soil against pile, best described as a skin friction angle, δ, depends upon soil type, pile material, and surface texture. Potyondi (1961) has examined the frictional resistance of several pile-soil combinations and his values of δ are given in Table 3.2. The skin friction of piles in sand can be determined as follows:

$$f = \bar{p}_h \tan \delta \tag{3.14}$$

or

$$f = K_s \bar{p}_v \tan \delta \tag{3.15}$$

For depths less than the critical depth,

$$f = K_s\ \bar{\gamma}\ z \tan \delta \tag{3.16}$$

and for depths equal to or greater than critical

$$f = K_s\ \bar{\gamma}\ z_c \tan \delta \tag{3.17}$$

TABLE 3.2

Proposed coefficients of skin friction between soils and construction materials

[$f\phi = \delta/\phi$, $fc = \frac{c_a}{c}$, $fc_{max} = \frac{c_a max}{c_{max}}$; without factor of safety]

Construction material			Sand		Cohesionless silt			Cohesive granular soil		Clay		
			$0{\cdot}06 < D <$ 2·0 mm		$0{\cdot}002 < D < 0{\cdot}06$			50% Clay + 50% Sand		$D \leqq 0{\cdot}06$ mm		
	Surface finish of construction material		Dry	Sat.	Dry	Sat.		Consist. I. =1·0–0·5		Consist. Index: 1·0–0·73		
			Dense		Dense	Loose	Dense					
			$f\phi$	$f\phi$	$f\phi$	$f\phi$	$f\phi$	$f\phi$	fc	$f\phi$	fc	fc_{max}
Steel	Smooth	Polished	0·54	0·64	0·79	0·40	0·68	0·40	—	0·50	0·25	0·50
Steel	Rough	Rusted	0·76	0·80	0·95	0·48	0·75	0·65	0·35	0·50	0·50	0·80
Wood	Parallel to grain		0·76	0·85	0·92	0·55	0·87	0·80	0·20	0·60	0·4	0·85
Wood	At right angles to grain		0·88	0·89	0·98	0·63	0·95	0·90	0·40	0·70	0·50	0·85
Concrete	Smooth	Made in iron form	0·76	0·80	0·92	0·50	0·87	0·84	0·42	0·68	0·40	1·00
Concrete	Grained	Made in wood form	0·88	0·88	0·98	0·62	0·96	0·90	0·58	0·80	0·50	1·00
Concrete	Rough	Made on adjusted ground	0·98	0·90	1·00	0·79	1·00	0·95	0·80	0·95	0·60	1·00

after Potyondi

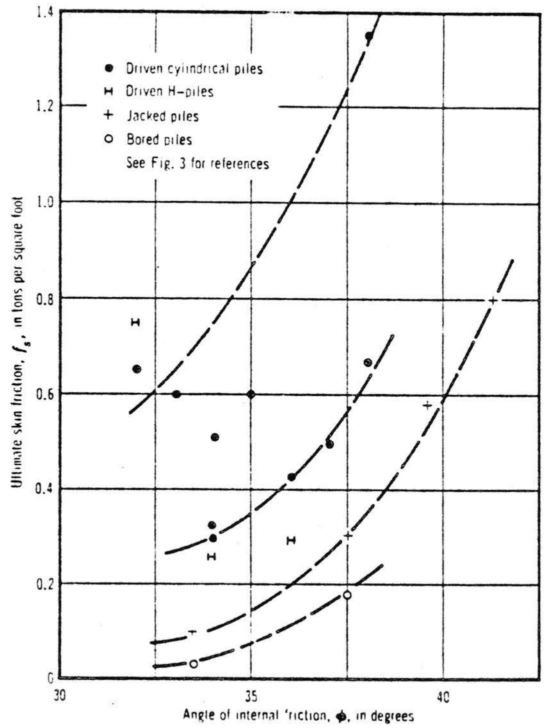


Figure 3.5. Ultimate Skin Friction of Piles in Sand.

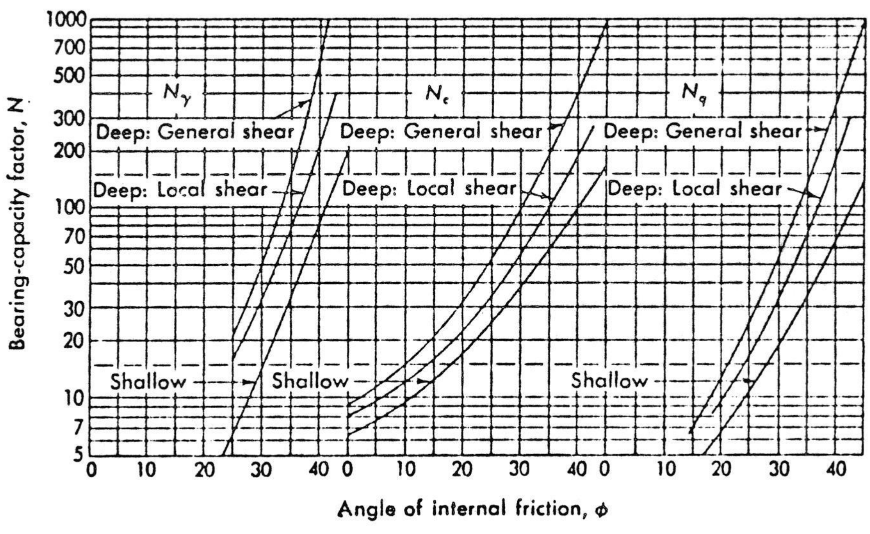


Figure 3.6. Bearing Capacity Factors for Shallow and Deep Square or Cylindrical Foundations.

Limiting values of skin friction presented by Meyerhof (1976) are given in Figure 3.5.

The end bearing component of pile capacity, Q_{EB}, can be determined by the general bearing capacity equation, using factors appropriate for deep foundations.

$$Q_{EB} = q_{ult} A_t = (cN_{cp} + \overline{p}_v N_{qp} + \tfrac{1}{2} \overline{\gamma} D N_{\gamma p}) A_t \qquad (3.18)$$

where:

q_{ult} = ultimate tip bearing capacity

A_t = area of pile tip

c = cohesion in the vicinity of the tip

$\overline{\gamma}$ = effective unit soil weight in the vicinity of the tip

D = pile diameter or width

N_{cp}, N_{qp}, $N_{\gamma p}$ = deep foundation bearing capacity factors (See Figure 3.6)

Since D is usually small, the $N_{\gamma p}$ term is often neglected. For piles in cohesionless soils (c = 0), the end bearing may be determined by the following expression:

$$Q_{EB} = \overline{p}_v N_{qp} A_t \qquad (3.19)$$

For cohesive soils ($\phi = 0$, $N_{qp} = 1$), the end bearing becomes:

$$Q_{EB} = (c N_{cp} + \overline{p}_v) A_t \qquad (3.20)$$

The concept of critical depth should be applied in determining $\overline{p}_v$ for cohesionless soils but should not be applied in the case of cohesive soils.

Soil properties required by the analysis described above may be measured by laboratory tests on indisturbed samples or may be inferred from the results of field tests such as the quasi-static cone penetration test, or the vane shear test. Figure 3.7 shows an approximate relation between quasi-static cone penetration resistance and the friction angle of sand.

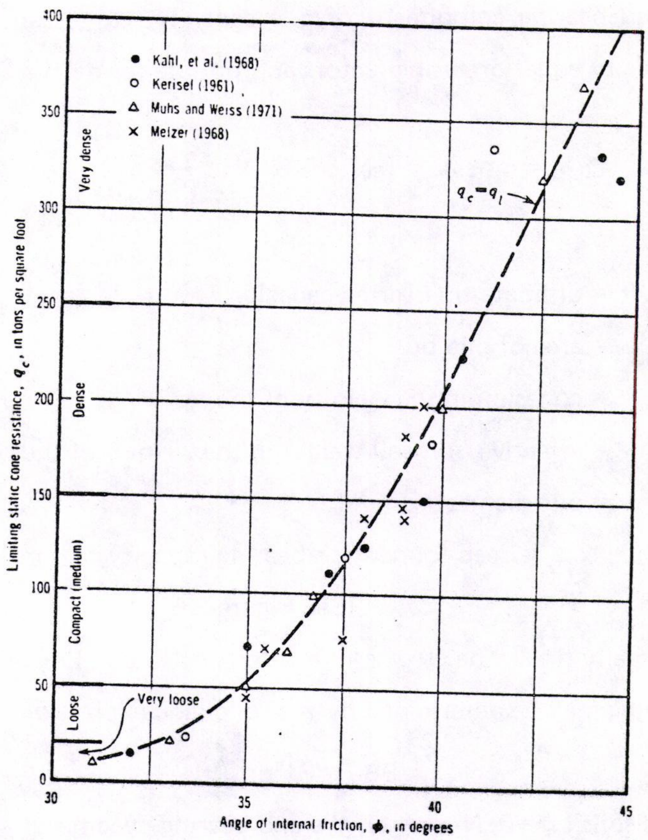


Figure 3.7. Approximate Relation between Limiting Static Cone Resistance and Friction Angle of Sand.

The following examples will serve to illustrate design by the limit equilibrium method.

EXAMPLE 1

Stratum	Depth (ft.)	Soil Type	$\bar{\gamma}$ (pcf)	c (psf)	ϕ	D_r
1	0-10	Stiff clay	118	1500	0°	—
2	10-17	Medium clay	61	850	0°	—
3	17-45	Very still clay	66	3000	0°	—

Water table at 10 ft.

Find the ultimate capacity of a 16-in. square concrete pile driven to a penetration of 35 ft.

$Q_{ult} = Q_{SF} + Q_{EB}$

$Q_{SF} = f_{avg}\ PL$

$P = (4)\dfrac{(16)}{12} = 5.33$ ft. (perimeter)

$L = 35$ ft. (length)

for Stratum 1, $\alpha = 0.55$ (from Fig. 3.4)

$f_1 = (1500)\ (0.55) = 825$ psf

for Stratum 2, $\alpha = 0.78$

$f_2 = (850)\ (0.78) = 663$ psf

for Stratum 3, $\alpha = 0.35$

$f_3 = (3000)\ (0.35) = 1050$ psf

$$f_{avg} = \frac{(10)\ (825) + (7)\ (663) + (18)\ (1050)}{35} = 908 \text{ psf}$$

$Q_{SF} = (908)\ (5.33)\ (35) = 169{,}400$ lbs.

$Q_{EB} = q_{ult}\ A_t$

$q_{ult} = cN_{cp} + p_v$

$N_{cp} = 9$ (from Fig. 3.6)

$q_{ult} = (3000)\ (9) + \quad (10)\ (118) + (7)\ (61) + (18)\ (66) \quad = 29{,}800$ psf

$A_t = \dfrac{(16)\ (16)}{144} = 1.78$ sq. ft.

$Q_{EB} = (29{,}800)\ (1.78) = 53{,}000$ lbs.

$Q_{ULT} = 169.4 + 53 = \underline{222.4 \text{ kips}}$

EXAMPLE 2

Soil is a uniform deposit of loose sand with $\overline{\gamma}$ = 60 pcf, c = 0, ϕ = 30°, D_r = 30%. The water table is at the ground surface. Find the ultimate capacity of a 16-in. square concrete pile driven to a penetration of 35 ft.

$Q_{ULT} = Q_{SF} + Q_{EB}$

$Q_{SF} = f_{avg} PL$

P = 5.33 ft.

L = 35 ft.

$z_c = 10D = 10 \left[\frac{16}{12}\right] = 13.33$ ft.

K_s = 2 to 3 say 2.5 (from Table 3.1)

$\delta = 0.80\,\phi = 24°$ (from Table 3.2)

For depths less than the critical depth,

$f = K_s \overline{\gamma} z \tan \delta$

$f = (2.5)\ (60)\ (\tan 24°)z = 66.78z$

For depths greater than or equal to the critical depth,

$f = K_s \overline{\gamma} z_c \tan \delta$

$f = 66.78z_c = (66.78)\ (13.33) = 890$ psf

$$f_{avg} = \frac{(13.33)\ (890)\ (0.5) + (21.67)\ (890)}{35} = 720 \text{ psf}$$

$Q_{SF} = (720)\ (5.33)\ (35) = 134{,}300$ lbs.

$Q_{EB} = q_{ult} A_t$

$q_{ult} = p_v N_{qp}$

$\overline{p}_v = \overline{\gamma} z_c = (60)\ (13.33) = 800$ psf

N_{qp} = 33 (from Fig. 3.6)

$q_{ult} = (800)\ (33) = 26{,}400$ psf

A_t = 1.78 sq. ft.

$Q_{EB} = (26{,}400)\ (1.78) = 47{,}000$ lbs.

$Q_{ULT} = 134.3 + 47.0 = \underline{181.3 \text{ kips}}$

EXAMPLE 3

Stratum	Depth (ft.)	Soil Type	$\bar{\gamma}$ (pcf)	c (psf)	ϕ	D_r
1	0- 7	Stiff clay	120	1800	0°	–
2	7-12	Dense Silt	56	0	28°	>70%
3	12-22	Stiff Clay	60	1000	0°	–
4	22-33	Firm Sand	64	0	35°	50%
5	33-50	Very Dense Sand and Gravel	70	0	42°	>70%

Water table at 7 ft.

Find the ultimate capacity of a 16-in. square concrete pile driven to a penetration of 35 ft.

$Q_{ULT} = Q_{SF} + Q_{EB}$

$Q_{SF} = f_{avg}\, PL$

P = 5.33 ft.

L = 35 ft.

for Stratum 1, α = 0.45 (from Fig. 3.4)

f_1 = (1800) (0.45) = 810 psf

for Stratum 2, z_c = 20D = 26.67 ft., $\therefore z < z_c$

K_s = 3 to 5 say 3 (from Table 3.1)

$\delta = 0.87\,\phi = 24.4°$ (from Table 3.2)

at z = 7 ft. (top of stratum)

$f = K_s\,\bar{\gamma}\,z \tan\delta$ = (3) (120) (7) (tan 24.4°) = 1143 psf

at z = 12 ft. (bottom of stratum)

$f = (3)\,[(120)(7) + (56)(5)]\,(\tan 24.4°) = 1524$ psf

$$f_2 = \frac{1143 + 1524}{2} = 1333 \text{ psf}$$

for Stratum 3, α = 0.70

f_3 = (1000) (0.70) = 700 psf

for Stratum 4, z_c = 15D = 20 ft., $\therefore z > z_c$

K_s = 3 to 5 say 3 (from Table 3.1)

$\delta = 0.80\,\phi = 28°$ (from Table 3.2)

$f_4 = K_s\,\bar{\gamma}\,z_c \tan\delta$ = (3) (64) (20) (tan 28°) = 2042 psf

for Stratum 5, z_c = 20D = 26.67 ft., $\therefore z > z_c$

K_s = 3 to 5 say 3 (from Table 3.1)

$\delta = 0.80\,\phi = 33.6°$ (from Table 3.2)

$f_5 = K_s \bar{\gamma} z_c \tan \delta = (3)(70)(26.67)(\tan 33.6°) = 3721$ psf

$$f_{avg} = \frac{(7)(810) + (5)(1333) + (10)(700) + (11)(2042) + (2)(3721)}{35} = 1407 \text{ psf}$$

$Q_{SF} = (1407)(5.33)(35) = 262{,}000$ lbs.

$Q_{EB} = q_{ult} A_t$

$q_{ult} = \bar{p}_v N_{qp}$

$N_{qp} = 90$ (use shallow foundation factor because of small penetration, i.e., $< 4D$)

$\bar{p}_v = \bar{\gamma} z_c = (70)(26.67) = 1867$ psf

$q_{ult} = (1867)(90) = 168{,}000$ psf

$Q_{EB} = (168{,}000)(1.78) = 299{,}000$ lbs.

$Q_{ULT} = 262 + 299 =$ <u>561 kips</u>

LOAD TRANSFER FUNCTION METHOD

Analysis of the load-deformation behavior of piles may be accomplished by using a load transfer function approach or by using an axisymmetric finite element analysis. In certain cases, an elastic solid analysis based on the Mindlin equations could be used.

In the load transfer function analysis, the pile is treated as a deformable member, the stress-displacement relationships for skin friction and end bearing are considered and may exhibit non-linear behavior, and the peak values of skin friction and end bearing are not required to occur simultaneously. It is assumed that loads transferred to the soil do not affect existing lateral or vertical stresses.

This method of analysis requires that the pile be divided into segments and a load transfer curve showing developed skin friction vs. displacement be developed for each segment. (See Figure 3.8). A tip load vs. tip displacement curve is also required. To compute the

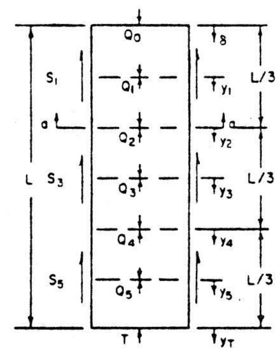


Figure 3.8a. Axially Loaded Pile Divided into Three Segments

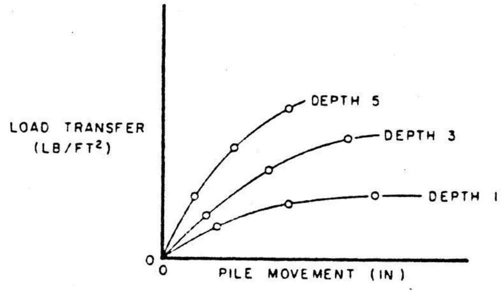


Figure 3.8b. Typical Curve Showing Load Transfer Versus Pile Movement

load-settlement curve for the top of the pile, the solution proceeds through the following steps (Coyle and Reese, 1966):

1. Assume a small tip movement.
2. Determine the tip load corresponding to the assumed tip movement.
3. Estimate the midpoint movement of the bottom segment.
4. From the appropriate load transfer curve, determine the load transferred to the soil through skin friction.
5. The load at the top of the bottom segment is equal to the tip load plus the skin friction load.
6. Use the average load in the pile segment and compute the elastic deformation at the midpoint of the segment.
7. Compute a value for movement of the midpoint of the segment by adding the elastic deformation at the midpoint to the movement of the bottom of the segment (the tip, in this case).
8. If the computed movement does not agree with the assumed movement within a specified tolerance, repeat steps 4 through 7 until convergence is achieved.
9. Go to the next segment above and repeat the process until the top load and displacement have been determined.
10. Repeat this procedure using different assumed tip movements until enough points have been determined to adequately define the load-settlement curve.

Load transfer curves for clay, described by Coyle and Reese (1966), are shown in Figure 3.9. The curves for sand shown in Figure 3.10 are suggested by Coyle and Sulaiman (1967). The soil shear strength used in Figure 3.10 is based upon the assumption that the lateral pressure coefficient is constant with depth and is equal to one.

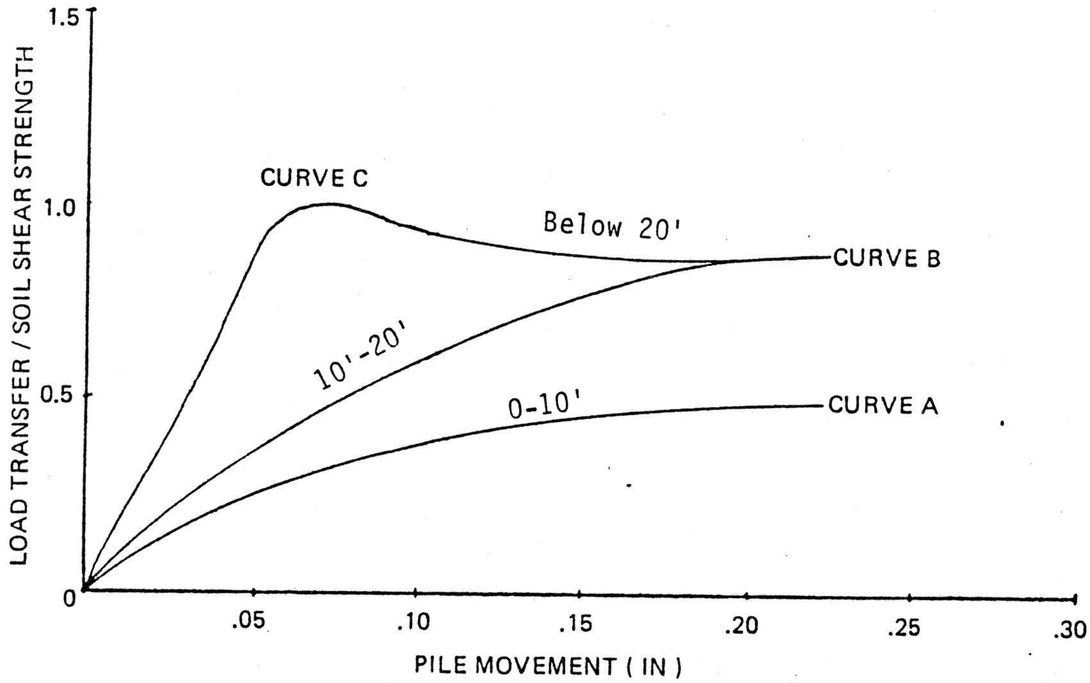


Figure 3.9 Load Transfer Curves for Clay

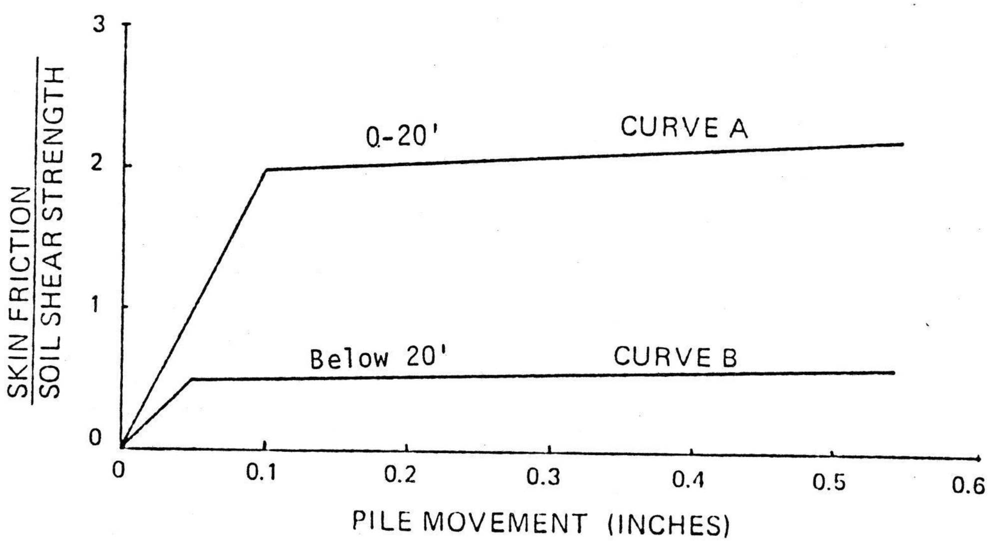


Figure 3.10. Load Transfer Curves for Sand

The tip load vs. tip movement curves for piles bearing in clay are based upon work done by Skempton (1951). The relationship can be estimated from the following equation.

$$\frac{P}{B} = \frac{4}{E/c} \cdot \frac{q}{q_{ult}} \qquad (3.21)$$

where

P = tip settlement

B = tip width or diameter

E = secant modulus of the clay at a ratio of applied stress to ultimate stress to ultimate stress of q/q_{ult}

c = cohesion

q = tip bearing pressure

q_{ult} = ultimate bearing capacity of the tip

This can be related to compression test results by the equation

$$\frac{P}{B} = 2\varepsilon \qquad (3.22)$$

where

ε = strain in compression test at a ratio of applied stress to ultimate stress of q/q_{ult}.

The load-deformation behavior of piles bearing in sand is difficult to predict. Some typical values of ultimate tip resistance and tip resistance vs. tip displacement given by Reese (1978) for drilled shafts bearing in sand are given in Figures 3.11 and 3.12.

ELASTIC SOLID ANALYSIS

An elastic solid analysis treats the soil as a homogeneous, isotropic elestic half-space. Stresses and deformations can be computed by applying the Mindlin (1936) equations for a point load applied in the interior of an elastic half-space. An interative solution process can be used to obtain an approximate solution for non-linear soil properties. Most elastic solutions give a reasonable approximation of stresses within the soil mass but do not give deformations that are compatible with observed values. Figure 3.13 gives influence values for stresses due to uniform skin friction and point load as computed by the Mindlin equations.

FINITE ELEMENT ANALYSIS

The finite element method has provided the means for assessing the behavior of pile-soil systems that cannot be analyzed by any other available method. To perform an analysis by this method, the pile and soil are subdivided into a series of small elements of finite size which are connected only at discrete points (nodal points), usually at the corners of the elements. The constitutive properties of each element are specified and may be non-linear. Since each element may have different properties, this method is suited for the solution of problems involving layered systems, piles with abrupt changes in cross-section, discontinuous soil stratification, and many other cases where exact theoretical solutions are not available. For details of the solution process by the finite element method, reference should be made one of the many texts on the subject.

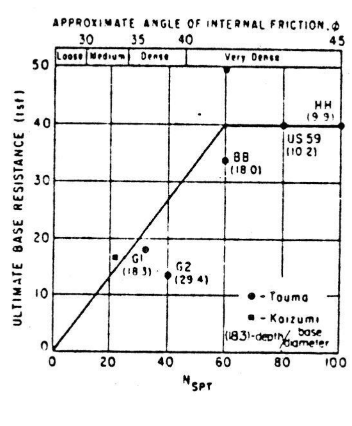


Figure 3.11. Ultimate Base Resistance in Sand Versus N_{SPT}

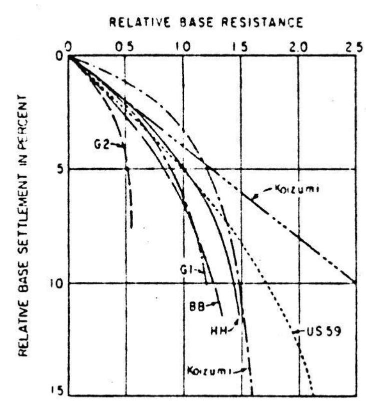
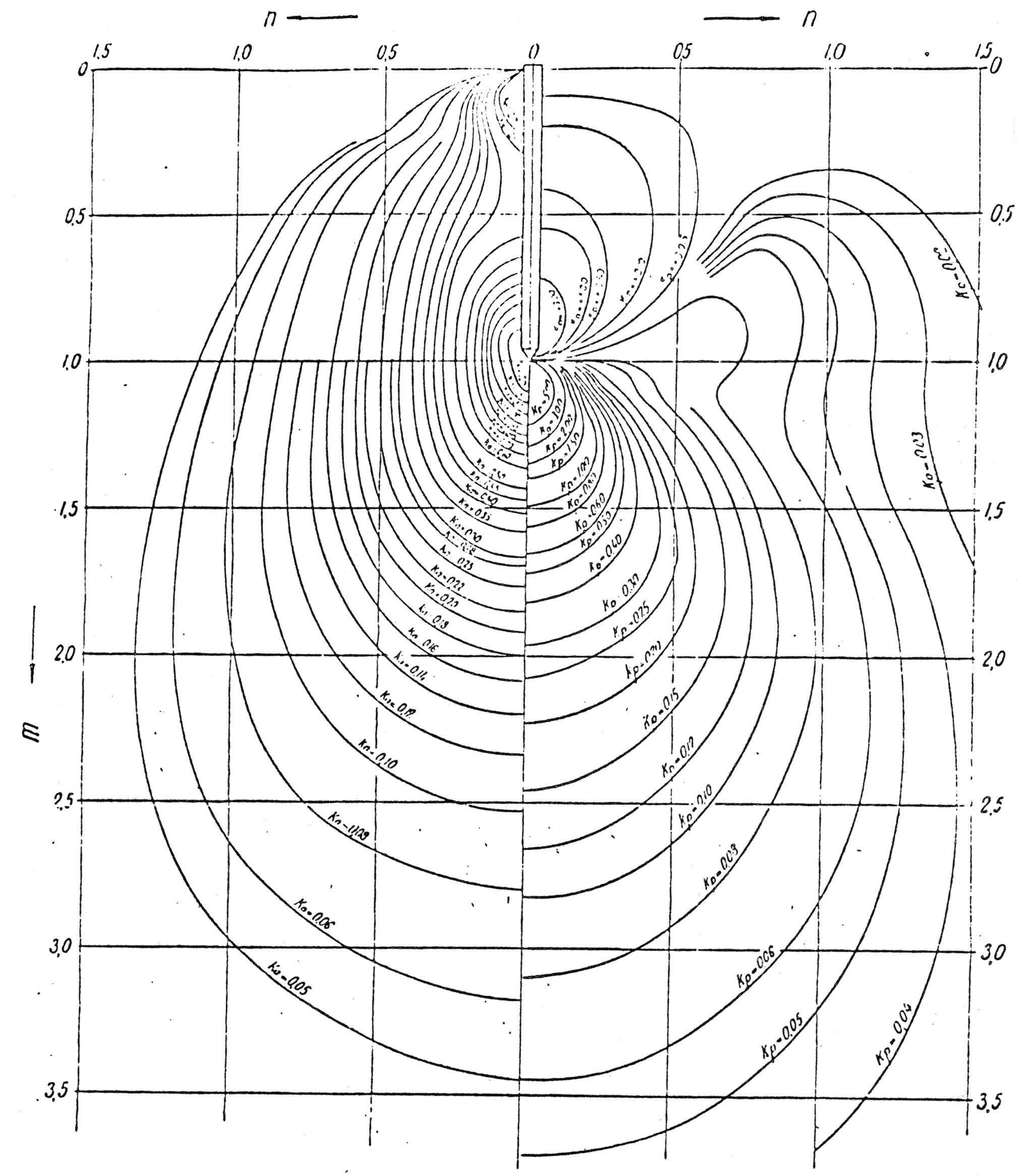


Figure 3.12. Relative Base Resistance in Sand Versus Relative Base Settlement



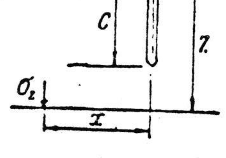
Influence curves for vertical stresses due to uniform skinfriction along the pile

(All values are negative or compression)
Poisson's modulus $\mu = 1/2$

$$\sigma_z = \frac{P_s}{C^2} \times K_o$$

$$m = \frac{z}{c}$$

$$n = \frac{x}{c}$$



Influence curves for vertical stresses due to pile point-load

(Kp values preceded by (+) means tension)
Poisson' modulus $\mu = 1/2$

$$\sigma_z = \frac{P_p}{C^2} \times K_p$$

$$m = \frac{z}{C}$$

$$n = \frac{x}{C}$$

Figure 3.13. Influence Values Based Upon Mindlin Equations

FIELD LOAD TESTS

The most common field load test is a compressive load test of a single pile. Occasionally load tests of drilled shafts are performed, but the high load-carrying capacity of this type of foundation element requires hefty loading and reaction systems and thus the test becomes expensive. A pile load test measures the ultimate capacity of a single pile at the time of loading. Deformation observed during the test will give an indication of the behavior of the pile under short-term loading. No other method can provide this information with equal accuracy.

The capacity and behavior of pile groups cannot be determined from tests on single piles, nor can long-term deformations be determined from short-term tests. Another factor which must be considered is the possibility of downdrag or negative skin friction developing when a pile penetrates a compressible clay layer. It may also be possible for piles driven through very loose sand to lose some skin friction due to a stress relaxation in the sand. Where the possibility of negative skin friction or stress relaxation developing exists, it is desirable to separate the skin friction and end bearing components of pile capacity during pile load tests. At present, only two acceptable methods are available for this purpose. Load tests of piles which are instrumented to measure load distribution along the pile can separate skin friction and end bearing as can pulling tests performed after compressive loading tests.

There are many procedures for load-testing piles. The load test procedures most commonly used include the maintained load test, the Texas Quick Test, and the constant rate of penetration test. These procedures are described in the following paragraphs.

Maintained Load Test

Load tests using the maintained load (ML) test procedure may be either proof tests to verify pile capacity or failure tests to determine the ultimate capacity of the pile. Failure tests will allow the designer to work to a selected factor of safety and optimize his design. The actual factor of safety cannot be determined from proof tests and may be considerably higher than is required for a conservative but economical design.

In the ML test procedure, loads are applied in increments, and each increment is maintained for a specified time or until the rate of settlement is less than a specified value. After the maximum load has been reached and maintained for the required time, the load is removed decrementally at specified intervals. Movement of the top of the pile is recorded immediately before and after loading or unloading and at intervals while the load is maintained constant.

The ML test procedure required by Arkansas Highway Department Standard Specifications (1972) calls for loading the test pile to 200 percent of the design load in increments of 25 percent of the design load. Increments are added at 30 minute intervals with settlement readings taken immediately before and after the addition of each load increment and three times between load increments. The unloading of the pile is accomplished by three decrements of 25 percent of the applied load, a decrement of 15 percent of the applied load and a final decrement of 10 percent of the applied load. The decrements are removed at 30 minute intervals with rebound readings taken before and after each decrement. A final rebound reading is taken 12 hours after the entire test load has been removed.

Texas Quick Test

In the Texas quick (TQ) test procedure, the load increments are the same as for the ML test but are applied at intervals of two and one-half minutes. Settlement readings are taken immediately before and after each load increment. When the untimate load is reached, loading is stopped and the load and settlement are allowed to stabilize. Load and settlement readings are taken at two and one-half and five minutes after loading is stopped. The entire load is then removed and rebound readings are taken immediately, and at two and one-half and five minutes after removal of the load.

Constant Rate of Penetration Test

The constant rate of penetration (CRP) test procedure was proposed by Whitaker and Cooke (1961). In this test, load is applied to the pile in a manner to achieve a constant rate of penetration of the pile into the soil. The rates of penetration recommended by Whitaker and Cooke (1961) are 0.03 inches per minute for cohesive soils and 0.06 inches per minute for cohesionless soils although they report that rates may vary from half to twice these values without significantly affecting the results. Simultaneous readings of load and settlement or rebound are taken during loading and unloading.

Interpretation of Results

After the load-settlement relationship is determined, the failure load must be established. There is no universally accepted criterion for establishing failure, but it is generally accepted that both load and settlement should be considered. Chellis (1961) has summarized 17 different criteria as follows:

1. The test load shall be twice the contemplated design load and shall be maintained constant for at least 24 hr and until settlement or rebound does not exceed 0.22 in. in 24 hr. The design load shall not exceed one-half the maximum applied load provided the load-settlement curve shows no signs of failure and the permanent settlement of the top of the pile, after completion of the test, does not exceed ½ in. (Boston Building Code).
2. Observe the point at which, no settlement have occurred for 24 hr, the <u>total</u> settlement including elastic deformation of the pile is not over 0.01 in. per ton of test load, and divide by a factor of safety of 2 (Department of Public Works, State of California).
3. The safe allowable load shall be considered as 50 percent of that load which, after a

continuous application for 48 hr, produces a permanent settlement not greater than ¼ in. measured at the top of the pile. This maximum settlement shall not be in-increased by continuous application of the test load for 60 hr or longer (AASHO).

4. Observe the point at which the plastic curve breaks sharply, and divided by a factor of safety of 1.5.
5. Tests shall be made with 200 percent of the proposed load, and considered unsatisfactory if, after standing 24 hr, the total net settlement after rebound is more than 0.01 in. per ton test load (building laws of the City of New York).
6. Observe the point at which the gross settlement begins to exceed 0.03 in. per ton of additional load, and divide by a factor of safety of 2 for static loads or 3 for vibratory loads (W.H. Rabe, Design Engineer, Bureau of Bridges, State of Ohio).
7. Draw tangent lines to the general slopes of the upper and lower portions of the curve, observe the load at their intersection, and divide by a factor of safety of 1.5 or 2.
8. Observe the point at which the slope of the curve of gross settlement is four times the slope of the graph of elastic deformation of the pile, and divide by a suitable factor of safety.
9. The allowable axial load on an isolated pile shall not exceed: (a) 50 percent of the yield point under test load. The yield point shall be defined as the point at which an increase of load produces a disproportionate increase in settlement; or (b) one-half of the load which causes a net settlement, after deducting rebound, of 0.01 in. per ton of test load, which has been applied for a period of at least 24 hr; or (c) one-half of that load under which, during a 40-hr period of continuous load applications, no additional settlement takes place (optional rules of International Conference of Building Officials Uniform Building Code).
10. Take two-thirds of the maximum test load in a case where settlement is not excessive and where load and settlement were proportionate and the curve remained a straight line. Where the test load was carried to failure, take two-thirds of the greatest load at which settlement was not excessive and at which loads and settlements were proportionate (United States Steel Co.).
11. With several consistent tests over the area of the structure, take from one-half to two-thirds of the failure load, considered as somewhere in the vicinity of the break in the curve showing increased settlement per unit of load added (Bethlehem Steel Co.).

12. The safe allowable load shall be considered as 50 percent of that load which, after a 48-hr application, causes a permanent settlement of not more than ¼ in. (New York State Department of Public Works).
13. One-half of the test load shall be allowed for the carrying load, if the test shows no settlement for 24 hr and the total settlement does not exceed 0.01 in. multiplied by the test load in tons (Chicago Building Code).
14. Observe the load at which is produced an increase in settlement disproportionate to the increase in load, and apply a factor of safety of 2 (Los Angeles Building Code).
15. Observe the load carried without exceeding a total permanent settlement of ¼ in. in 48 hr and divide by a factor of safety of 2 (Louisiana Department of Highways).
16. For important permanent structures, take the safe load on well-driven timber and concrete piles, with a final set of, say, ten blows to 1 in. at one-half to two-thirds of the test load which produces a final settlement gradually of ½ in. after a period of 10 days' rest. For well-placed undriven concrete piles, tested to twice their estimated bearing capacity, the safe bearing load has been taken in practice at one-half the test load which gives a settlement of 3/8 in. after a period of rest of 10 days (W. Simpson, "Foundations," Constable & Co., Ltd., London, 1928).
17. Observe the point at which the gross settlement begins to exceed 0.05 in. per ton of additional load, or at which the plastic settlement begins to exceed 0.03 in. per ton of additional load, and divided by a factor of safety of 2 for static loads or 3 for vibratory loads (Dr. R.L. Nordlund, Raymond Concrete Pile Company).

The Texas Highway Department uses a combination of rules 7 and 17 for interpretation of the results of the Texas quick test. Details of the interpretation procedure are given below and in Figure 3.14.

1. Plot a graph of load versus gross settlement using any convenient scale.
2. Draw one line originating at the point of zero load and settlement and tangent to the initial flat portion of the gross settlement curve. (The slope of this line will be approximately the same as the slope of the recovery line).
3. Draw a second line tangent to the steep portion of the gross settlement curve with a slope of 0.05 in. of settlement per ton of load for a pile test and a slope of 0.01-in. per ton of load for a drilled shaft test.
4. The load at the intersection of the two tangents drawn in steps 2 and 3 is defined as the ultimate bearing capacity of the pile and will be used to establish a proven "maximum safe static" load.

5. The proven maximum safe static load for piling is defined as one-half of the ultimate bearing capacity obtained in step 4. The proven maximum safe static load for a drilled shaft is defined as one-half the ultimate bearing capacity obtained in step 4 provided the gross settlement at the proposed design load is not more than one-half inch.

(Figure 3.14)

PILE CAPACITY BASED UPON DRIVING RESISTANCE

Methods based upon driving resistance will usually fall into two categories: (1) methods based upon dynamic formulas equating the kinetic energy produced by the pile-driving hammer to the work done in advancing the pile plus the energy losses in the hammer-pile-soil system, and (2) methods based upon the one-dimensional wave equation describing the effects produced when a long slender rod is struck on its end.

Dynamic Formulas — The simplest dynamic formula is based upon the assumption that the pile is perfectly rigid and that no energy is lost during driving.

$$Wh = R_u s \qquad (3.23)$$

where:

W = weight of hammer

h = height of drop

R_u = ultimate pile capacity

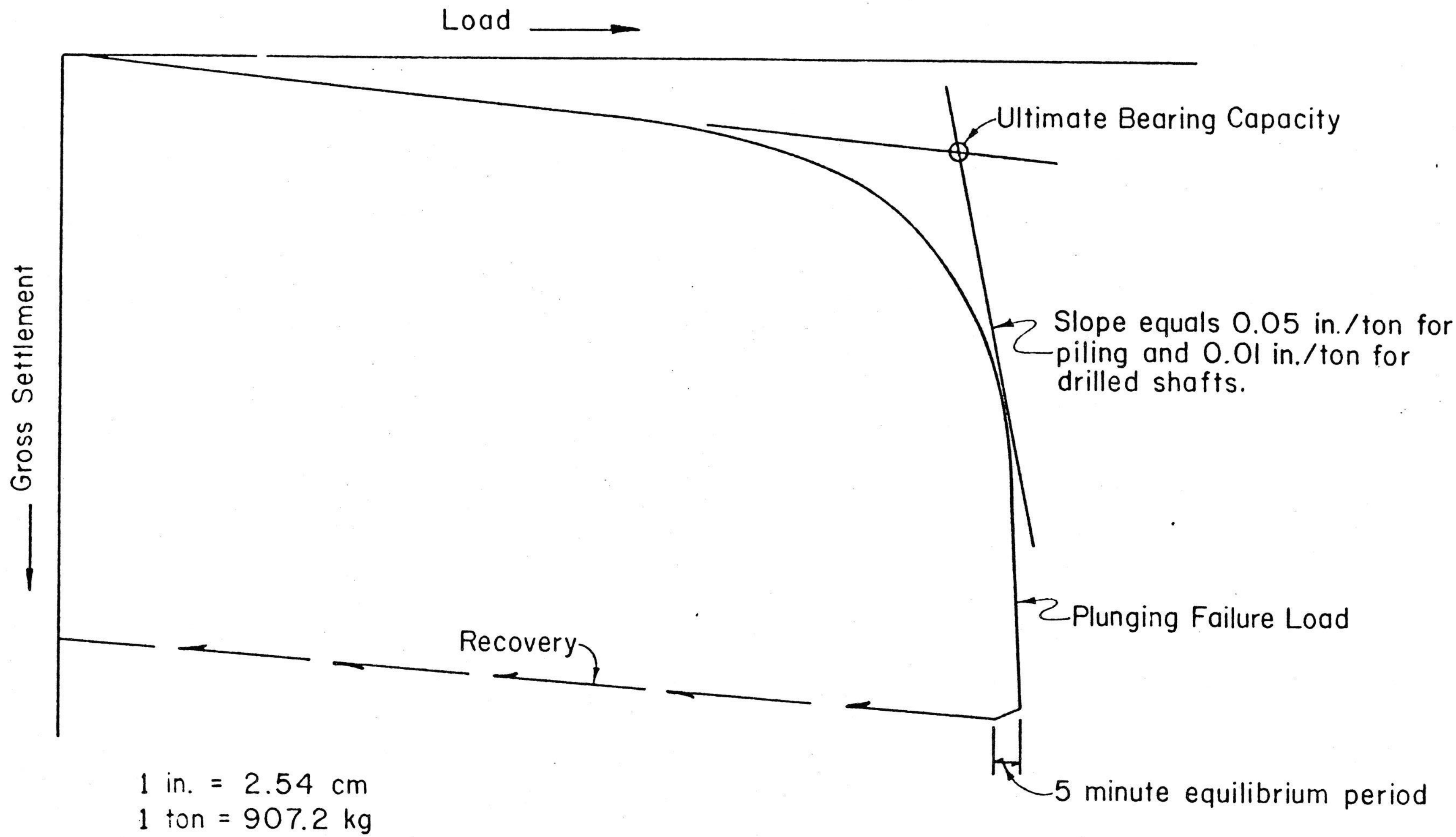


Figure 3.14. Typical Load Settlement Graph

s = set or penetration of the pile under the last blow

The weight, drop, and set can be measured and R_u can be determined. This equation (3.23) does not give reasonable values of R_u because there are significant energy losses in the hammer-pile-soil system. Energy is lost through friction in the hammer parts, impact, and elastic compression of the pile cap, pile, and soil. The primary difference between the various pile driving equations is the manner in which these losses are considered. For example, in the Engineering News Formula,

$$R_u = \frac{Wh}{s + c} \tag{3.24}$$

where

c = elastic compression of hammer-pile-soil system

the energy loss, $R_u c$, is dependent only upon the type of hammer used to drive the pile. For all types of piles and soils, c is assumed to be 1.0 inch for drop hammers and 0.1 inch for single-acting steam hammers. The Hiley formula is based upon a more realistic appraisal of energy losses. This formula is considered a comprehensive formula and is expressed as

$$R_u = \frac{e W_r h}{s + 0.5\,(c_1 + c_2 + c_3)} \cdot \frac{W_r + n^2 W_p}{W_r + W_p} \tag{3.25}$$

where

e = efficiency of pile hammer (ratio of energy output to energy rating)

$W_r h$ = energy rating of hammer (W_r = wt. of hammer, h = ht. of drop)

W_p = weight of pile

n = coefficient of restitution

c_1 = elastic compression of pile head and cap

c_2 = elastic compression of pile

c_3 = elastic compression of soil

The term $(W_r + n^2 W_p) / (W_r + W_p)$ is a treatment of energy loss during impact. The values of c_1, c_2 and c_3 may be estimated by using Tables 3.3, 3.4 and 3.5, or c_1, c_2 may be computed by the following expression:

$$c = \frac{R_u l}{AE} \tag{3.26}$$

where

c = elastic compression of cap (c_1) or pile (c_2)

TEMPORARY COMPRESSION ALLOWANCE C_1 FOR PILE HEAD AND CAP[a]

Material to which blow is applied	Easy driving, p_1 = 500 psi on cushion or pile butt if no cushion, in.	Medium driving, p_1 = 1,000 psi on head or cap, in.	Hard driving, p_1 = 1,500 psi on head or cap, in.	Very hard driving, p_1 = 2,000 psi on head or cap, in.
Head of timber pile	0.05	0.10	0.15	0.20
3-4-in. packing inside cap on head of precast concrete pile	0.05 + 0.07[b]	0.10 + 0.15[b]	0.15 + 0.22[b]	0.20 + 0.30[b]
½-1-in. mat pad only on head of precast concrete pile	0.025	0.05	0.075	0.10
Steel-covered cap, containing wood packing, for steel piling or pipe	0.04	0.08	0.12	0.16
3/16-in. red electrical-fiber disk between two ⅜-in. steel plates, for use with severe driving on Monotube pile	0.02	0.04	0.06	0.08
Head of steel piling or pipe	0	0	0	0

[a] Largely from A. Hiley, "Pile Driving Calculations with Notes on Driving Force and Ground Resistance," *The Structural Engineer*, vol. 8, July and August, 1930.[7] For a fuller discussion of the means of obtaining these values see this reference. For purpose of this article values represent average conditions and may be used.

[b] The first figure represents the compression of the cap and wood dolly or packing above the cap, whereas the second figure represents the compression of the wood packing between the cap and the pile head.

NOTE: Superior numbers (with or without letters) refer to the Bibliography, pp. 641*ff*., in which the material is organized by subject.

TABLE 3.3

TEMPORARY COMPRESSION VALUES OF C_2 FOR PILES

Type of pile	Easy driving, $p_2 = 500$ psi for wood or concrete piles, 7,500 psi for steel, net section, in.	Medium driving, $p_2 = 1{,}000$ psi for wood or concrete piles, 15,000 psi for steel, net section, in.	Hard driving, $p_2 = 1{,}500$ psi for wood or concrete piles, 22,500 psi for steel, net section, in.	Very hard driving, $p_2 = 2{,}000$ psi for wood or concrete piles, 30,000 psi for steel, net section, in.
Timber pile, based on value of $E = 1{,}500{,}000$. Proportion for other values of E given in Table VI[a]	$0.004 \times L^b$	$0.008 \times L^b$	$0.012 \times L^b$	$0.016 \times L^b$
Precast concrete pile ($E = 3{,}000{,}000$[a,c])	$0.002 \times L$	$0.004 \times L$	$0.006 \times L$	$0.008 \times L$
Steel sheet piling, Simplex tube, pipe pile, Monotube shell, Raymond steel mandrel[d] ($E = 30{,}000{,}000$)	$0.003 \times L$	$0.006 \times L$	$0.009 \times L$	$0.012 \times L$

[a] All other values in direct proportion to p_2 and inverse proportion to E.

[b] L should be considered as length to center of driving resistance, not necessarily full length of pile.

[c] May reach 6,000,000 for exceptionally good mix.

[d] When computing p_2 for a Raymond steel mandrel, it is suggested that the weight of the mandrel be divided by 3.4 × the effective length of pile in feet to obtain the average area.

TABLE 3.4

TEMPORARY COMPRESSION OR QUAKE OF GROUND ALLOWANCE C_3[a]

All values of p_3 to be taken on projected area of pile tips or driving points for end-bearing piles and piles of constant cross section; on gross area of pile at ground surface in case of tapered friction piles; and on bounding area under H piles

	Easy driving, $p_3 = 500$ psi, in.	Medium driving, $p_3 = 1{,}000$ psi, in.	Hard driving, $p_3 = 1{,}500$ psi, in.	Very hard driving, $p_3 = 2{,}000$ psi, in.
For piles of constant cross section[b,c]	0 to 0.10	0.10	0.10	0.10

[a] Largely from A. Hiley, "Pile Driving Calculations with Notes on Driving Force and Ground Resistance," *The Structural Engineer*, vol. 8, July and August, 1930.[7] For a fuller discussion of the means of obtaining these values see this reference. For purpose of this article values represent average conditions and may be used.

[b] It is recognized that these values should probably be increased in the case of piles with battered faces, but insufficient test data are available at present time to cover this condition.

[c] If the strata immediately underlying the pile tips are very soft, it is possible that these values might be increased to as much as double those shown.

TABLE 3.5

l = length or thickness of pile cap and packing for computing c_1 or effective length of pile for computing c_2

A = cross-sectional area

E = modulus of elasticity

A more reliable procedure for determining $c_2 + c_3$ is to attach a sheet of paper to the side of the pile and, as it is being driven, draw a pencil along a stationary horizontal support marking the paper. A sketch of the arrangement is shown in Figure 3.15a and a typical trace is shown in Figure 3.15b. From the trace, the set, s, and the elastic compression of pile and soil, $c_2 + c_3$, may be determined. If it is assumed that the energy loss is due only to compression of the pile, then, the Danish formula is obtained, with

$$R_u = \frac{W_r h}{s + 0.5\, s_e} \qquad (3.27)$$

where

s_e = elastic compression of the pile

and

$$s_e = \sqrt{\frac{2W_r\, h\, L}{AE}} \qquad (3.28)$$

where

L, A, and E = length, area, and modulus of elasticity of the pile

The following example will illustrate the determination of pile capacity by dynamic formulae.

EXAMPLE

Determine the ultimate capacity of a 10.75 in. OD steel pipe pile with a wall thickness of 0.365 in. and a length of 45 ft. The pile is driven into sand by a single-acting steam hammer with an energy rating of 20,000 ft-lbs. The ram weighs 6500 lbs. The elastic compression of pile head and cap (c_1) is determined to be 0.08 in. and the bounce ($c_2 + c_3$) is measured as 0.45 in. The set (s) is measured as 0.078 in. The coefficient of restitution is taken as 0.50 and the hammer efficiency is taken as 65%. The pile has a cross-sectional area of 11.9 sq. in. and weighs 40.48 lb. per ft.

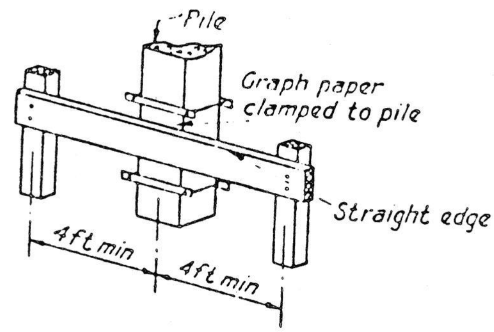


Figure 3.15a. Apparatus for Taking Readings on Pile

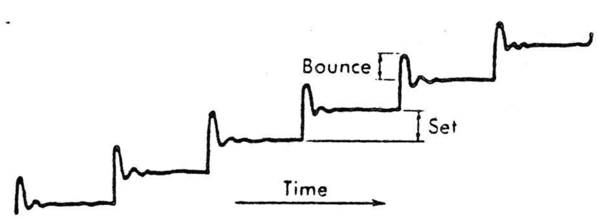


Figure 3.15b Diagram of Set and Temporary Compression

Hiley Formula.

$$R_u = \frac{e W_r h}{s + 0.5\,(c_1 + c_2 + c_3)} \cdot \frac{W_r + n^2 W_p}{W_r + W_p}$$

$$R_u = \frac{(0.65)\,(20{,}000)\,(12)}{0.078 + 0.5\,(0.08+0.45)} \cdot \frac{(6500) + (0.50)^2\,(45)\,(40.48)}{(6500 + (45)\,(40.48)}$$

$$R_u = 380{,}000 \text{ lbs.} = 190 \text{ tons}$$

Danish Formula.

$$R_u = \frac{W_r h}{s+0.5 s_e} \quad \text{and} \quad s_e = \sqrt{\frac{2 W_r h L}{AE}}$$

$$s_e = \sqrt{\frac{(2)\,(20{,}000)\,(12)\,(45)\,(12)}{(11.9)\,(30 \times 10^6)}} = 0.852 \text{ in.}$$

$$R_u = \frac{(20{,}000)\,(12)}{0.078 + 0.5\,(0.852)}$$

$$R_u = 476{,}000 \text{ lbs.} = 238 \text{ tons}$$

Engineering News Formula

$$R_u = \frac{W_r h}{s + 0.1}$$

$$R_u = \frac{(20{,}000)\,(12)}{0.078 + 0.1}$$

$$R_u = 1{,}348{,}000 \text{ lbs.} = 674 \text{ tons}$$

Wave Equation Methods. The wave equation describes the movement of stress waves in a long slender rod when it is struck on one end. This analysis was first applied to pile driving in the 1930's, but the tedious computations required inhibited its use. The development of high-speed digital computers and Smith's (1960) numerical solution of the wave equation have led to a fairly widespread use of this method of analysis. Two implementation packages presenting computer codes and documentation for application of the wave equation to pile driving are currently available (FHWA-IP-76-13, FHWA-IP-76-14). A different approach to the wave equation was taken by Goble and Rausche (1970). Transducers are attached to the pile near the top to measure the force and acceleration of the pile under a hammer blow. A small dedicated computer is used to determine the pile capacity from the transducer outputs.

In Smith's numerical solution of the wave equation, the hammer, pile and soil system are represented by a series of weights and springs (Figure 3.16). The cap block and anvil may also be depicted by weights and springs. The driving action is divided into small time elements of about .25 milliseconds and the pile is divided into segments of approximately 5 to 10 feet. In this manner, a reasonably accurate determination of pile stresses and penetration may be made for any particular system. The spring constants, K, are found for elastic material such as the pile and cap from the formula:

$$K = \frac{AE}{L} \tag{3.29}$$

where

A = cross sectional

E = modulus of elasticity

L = segment length

Soil resistance is found for skin friction as well as point bearing. The soil is treated as an elastic-plastic material with stress-strain relationship as shown in Figure 3.17. The ultimate elastic movement of the soil is termed the quake (Q).

As the pile moves a distance A, it develops the ultimate resistance R_u. Further movement does not increase resistance and the point will continue to B on Figure 3.17a. Elastic unloading then occurs following line BC until all forces are zero. The permanent set of the pile is then the distance OC = AB.

Side resistance is calculated identically as point bearing except there are separate values of quake and ultimate resistance for each segment. The side friction may be distributed over the side of the pile by varying the stress-strain relationships of the individual segments.

These values of soil resistance have not included the time effects as yet. The ground will offer more resistance to rapid motion than to slow motion. To account for this, Smith

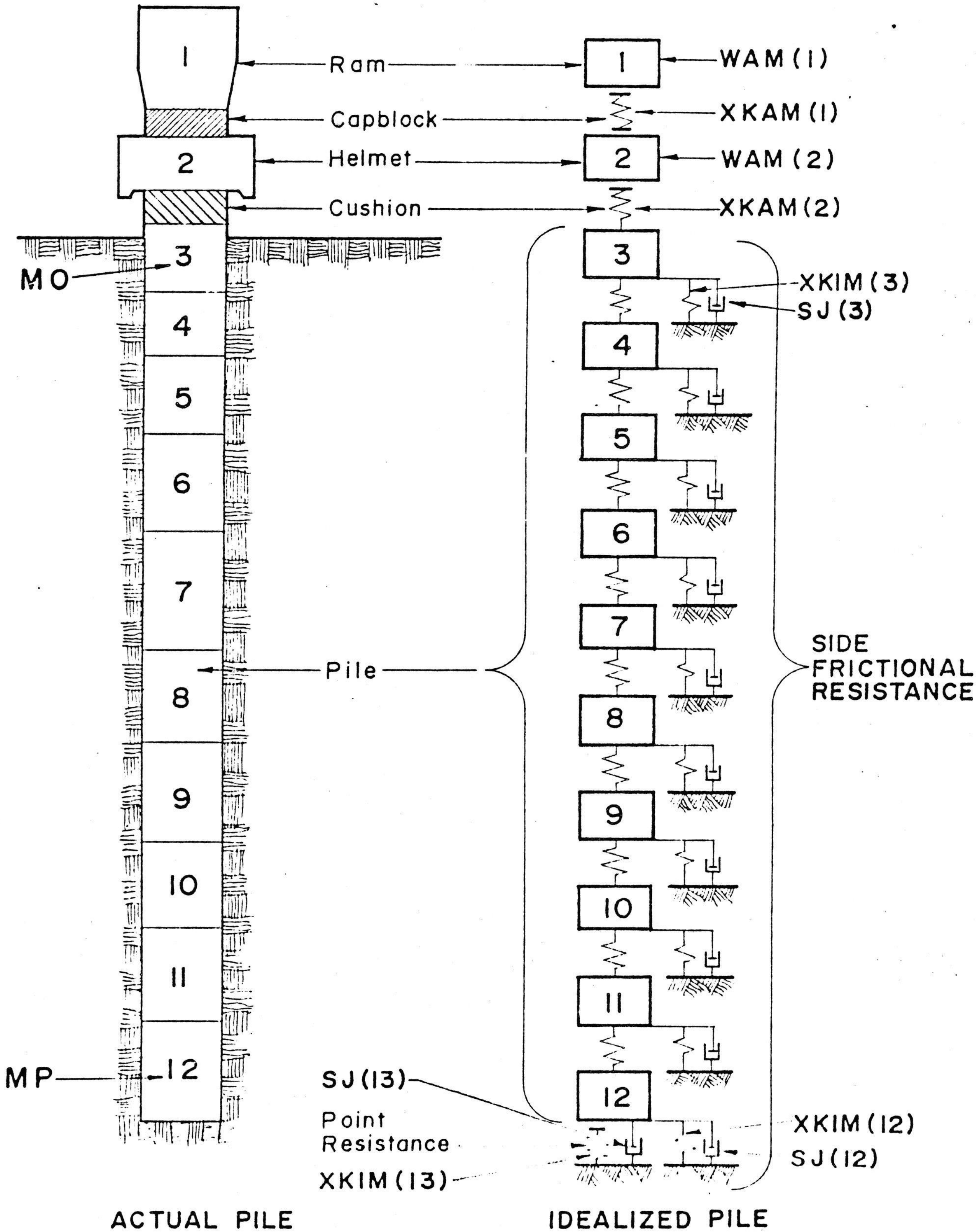


Figure 3.16. Idealization of a pile for purpose of analysis; Pile is divided into uniform concentrated weights and springs.

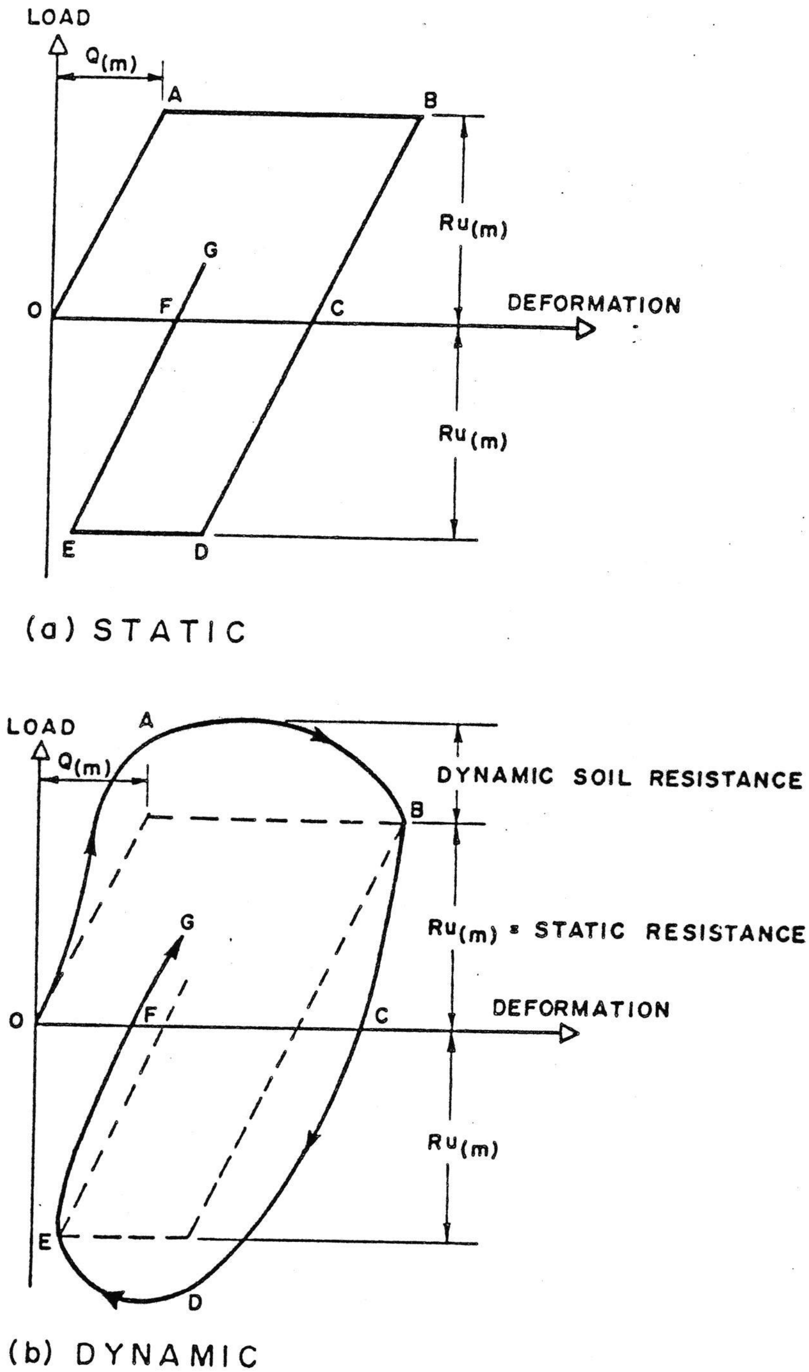


Figure 3.17. **Soil load-deformation characteristics.**

(1960) represented "viscous damping". The evaluation of the wave equation gives a velocity, v_p. By applying a damping constant, J_p, to the velocity, the product $J_p\, v_p$ increases ground resistance to account for damping. At any point X on the curve of Figure 3.17b, the instantaneous damping resistance is $J_p\, v_p\, R_x$. The total resistance of the pile to penetration is the static resistance plus the damping resistance.

The Case Western Reserve device uses a simple force balance method to relate dynamic measurements to a static capacity. The pile is assumed to be a rigid body struck by a time-varying hammer force (Goble and Rausche, 1970). Motion of the pile is resisted by a force, R, given by the expression

$$R(t) = R_0 + R_1\, v + R_2\, V^2 + R_3\, V^3 + \ldots \qquad (3.30)$$

where

V = the velocity of the pile

R_0 = static capacity

R_1, R_2, R_3 = constants

Using Newton's Second Law at the instant of zero velocity, the resistance is found to be:

$$R_0 = F\,(t_0) - m\, a(t_0) \qquad (3.31)$$

where

m = the mass of the pile

$a(t_0)$ = the acceleration at the time t_0 when the velocity is zero

$F(t_0)$ = the force at the top of the pile at the same time

A force transducer and an accelerometer are attached to the pile near the top to monitor force and acceleration for each blow of the pile hammer. A small field computer unit receives, records and analyzes the signals from the transducer and prints the computed pile capacity for each blow.

PILE GROUPS

The methods presented for determining pile capacity address the problem of the capacity of a single pile. Since piles are most often used in groups and seldom as individual piles, the difficult problem of assessing group capacity must be considered. The determination of group capacity can best be done by using a group efficiency factor defined as follows:

$$e = \frac{Q_g}{nQi} \tag{3.32}$$

where

e = efficiency

Q_g = capacity of pile group

Q_i = capacity of individual pile

n = number of piles in the group

Numerous formulas for determining group efficiency have been promulgated in the past, for example, the Converse - Labarre formula and Feld's One-Sixteenth Rule, but they seem to be based primarily on intuition and experimental verification is lacking. Field load tests of pile groups are very seldom performed because of the high loads and expense involved.

Vesic (1969) reported the results of model tests of groups of four-inch diameter piles in sand. The results of his tests indicated an efficiency of one for end bearing and an efficiency of approximately 1.3 for skin friction after making an allowance for the effect of the pile cap. For design of pile groups in sand, Vesic recommends that an efficiency of one be used.

Sowers, et al. (1961) compared the results of model tests of pile groups in clay with the block analysis. The block analysis of pile groups in clay is similar to the analysis of single piles in clay, with skin friction computed for the sides of the block enclosing the pile group and end bearing computed for the base of the block. The limiting value of efficiency is one.

$$Q_g = Q_{SF} + Q_{EB} \tag{3.33}$$

$$Q_{SF} = f_s (2X + 2Y) L \tag{3.34}$$

$$Q_{EB} = q_{ult} (X \cdot Y) \tag{3.35}$$

where

X = width of pile group

Y = length of pile group

L = embedded length of piles in group

The results of Sowers model tests are shown in Figure 3.18 and are expressed in the following equations.

$$e = 0.5 + \frac{0.4}{(n - 0.9)^{0.1}} \tag{3.36}$$

$$s_o = 1.1 + 0.4n^{0.4} \tag{3.37}$$

where

s_o = optimum spacing in pile diameters.

Although the efficiency at optimum spacing is slightly less than one, Sowers states that the error due to assuming an efficiency of one is inconsequential and recommends the use of an efficiency of one at pile spacing of optimum or greater.

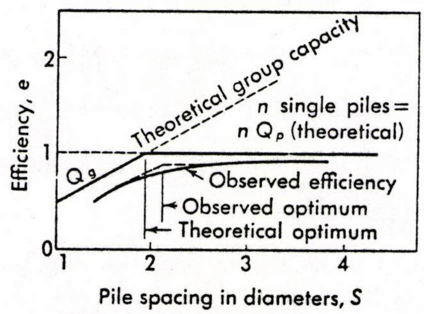


Figure 3.18. Efficiency of Friction Pile Groups in Saturated Clay

NEGATIVE SKIN FRICTION

It is not uncommon for piles to penetrate soft compressible strata before the tips reach a stratum of sufficient strength to support the load produced by the structure. If the effective stress in the compressible layer is increased, for example, by construction of an embankment, or by lowering the groundwater level, consolidation settlement will occur. When this settlement occurs, the soil in the compressible layer and all layers above will be moving downward with respect to the pile. The skin friction in the moving layers will add load to the pile and, if not considered in the design, may cause a failure of the foundation.

Because of the small movement required to develop maximum skin friction (on the order of 0.1 in.) the maximum skin friction should be computed (by the limit equilibrium method) for the compressible layer and all layers above and added to the load that must be carried by the pile. This means that only skin friction in layers below the compressible layer and end bearing of the tip are available to resist the structure load plus the negative skin friction.

SETTLEMENT OF DEEP FOUNDATIONS

The settlement of deep foundations can be separated into immediate settlement, due to "elastic" deformations of pile and soil, and long term settlement, due to consolidation of underlying compressible layers. Deep foundations in cohesionless soils generally exhibit only immediate settlements. Deep foundations in clay, however, will exhibit both immediate and long term settlements.

Immediate settlement can most conveniently be divided into three components: (1) axial compression of the pile, (2) deformations due to skin friction, and (3) deformations due to point load. These three elements form the basis of the load transfer function method described earlier. This method has been generally successful in predicting immediate settlements for piles in sand and in clay. The finite element method has also been used with some success in predicting immediate settlement.

Long term or consolidation settlement is computed in much the same manner as settlement of shallow foundation. The steps in this type of settlement analysis are:

1. Determine the initial effective stress in the compressible layers.
2. Determine the stress change due to the foundation load.
3. Using the initial effective stress and the final stress, determine the change in void ratio using the results of a consolidation test.
4. Compute the time-settlement relationship for each layer.
5. Combine the results of obtain a composite relationship for total settlement of the foundation.

The only step that is different for deep foundations in Step 2, the determination of stress change. One rule of thumb that has been widely used for friction pile groups is to assume that the total foundation load is applied at the lower third point of the piles over the gross area of the pile group. For end bearing piles, the load is assumed to act at the pile tips, over the gross area of the group. A better estimate of the stress change can be made by using the Mindlin solution or a finite element solution.

CHAPTER IV

RECOMMENDATIONS

The following procedures are recommended for implementation by the Arkansas State Highway and Transportation Department.

SITE INVESTIGATION

1. It is recommended that AHTD develop and maintain a comprehensive file of soil data for existing and planned bridge sites. This file should contain not only data generated by AHTD but also soil maps, geologic maps, and soil data from geotechnical consultants and other state and federal government agencies.

2. Preliminary field investigations should include seismic and resistivity surveys, where ever practical, as well as preliminary borings. The signal-enhancement type of seismograph is recommended.

3. The primary objectives of the detailed field investigation should be to define the soil stratification and to obtain high quality undisturbed samples of the foundation soil. The sampling tools recommended are the Shelby tube sampler, the Osterberg piston sampler, and either the Denison sampler or the Pitcher sampler.

4. If high quality undisturbed samples cannot be obtained, the in-situ properties should be assessed by using the quasi-static cone penetrometer. For soft to very soft clays, the field vane test is also appropriate.

5. For long term measurements of water levels or pore pressures, a double tube open system piezometer is recommended.

LABORATORY TESTING

1. The tests recommended for soil classification are the liquid limit, plastic limit and particle size analysis tests. The wet preparation procedure should be used for any soil containing clay.

2. Triaxial compression tests should provide the primary means to determine shear strength. For an undrained analysis involving homogeneous, intact, saturated clay, unconfined compression tests are acceptable.

3. To assess the stability of embankments constructed on clay shale, repeated direct shear tests should be performed on the clay shale and the residual strength used in the analysis.

4. One dimensional consolidation tests should be used to determine the compressibility of cohesive soils.

5. The use of back pressure saturation is encouraged for triaxial and consolidation tests. If accurate pore pressure measurements are to be made, the use of backpressure is essential.

DESIGN AND ANALYSIS

1. The general bearing capacity equation, with appropriate corrections for compressibility, shape, eccentricity, inclination of load, ground surface slope, and position of the water table should be used for determining the ultimate load capacity of shallow foundations.

2. Settlement should be estimated for each footing. The analysis of settlement of footings on cohesionless soils may be based upon empirical correlations.

3. Preliminary estimates of pile capacity should be done by the limit equilibrium method. These estimates should be verified at the time of installation by wave equation analyses or by a comprehensive dynamic formula such as the Hiley formula. Pile load tests should be performed on large jobs and in difficult soil conditions and the results correlated with the limit equilibrium analysis and driving resistance.

4. The immediate settlement of piles should be estimated by the load transfer function approach if no pile load test has been performed. Long term consolidation settlement should be estimated by using the Mindlin solution to determine stresses and one-dimensional consolidation test results to estimate the settlement.

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