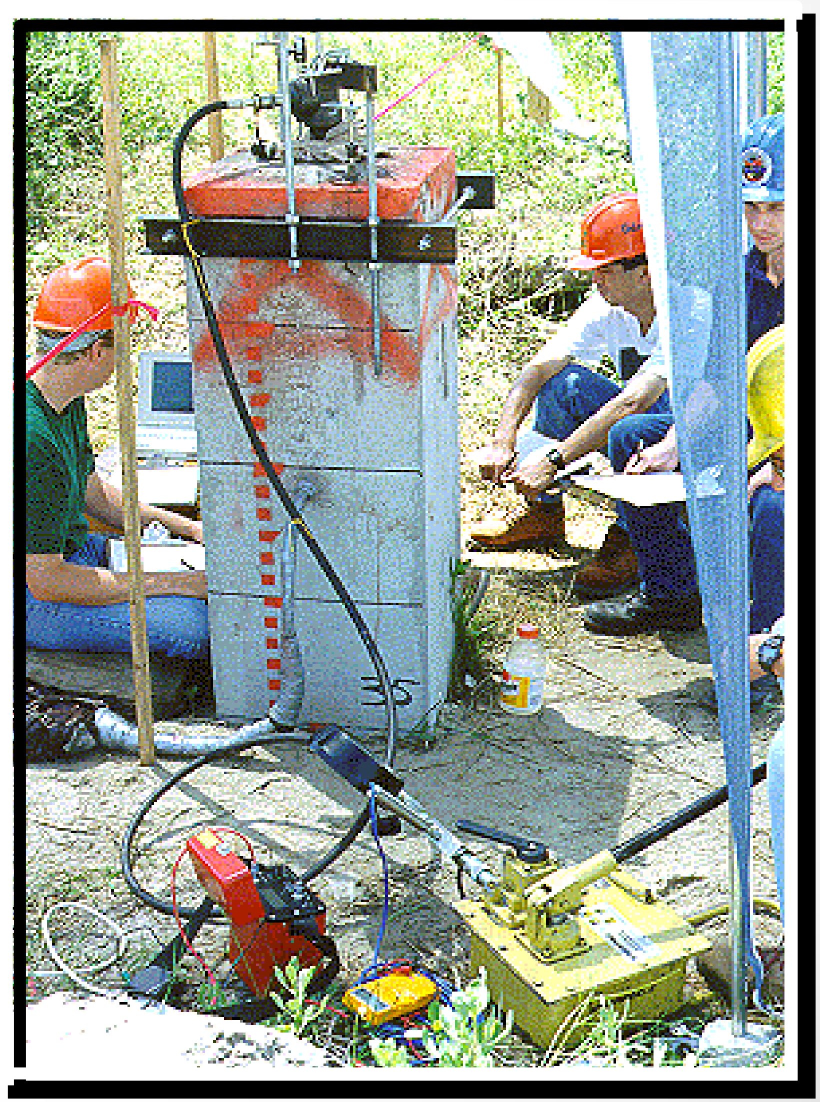
ENCE 4610
Foundation Analysis and Design

Lecture 15
Static Load Tests
Pile Settlement
Pile Groups

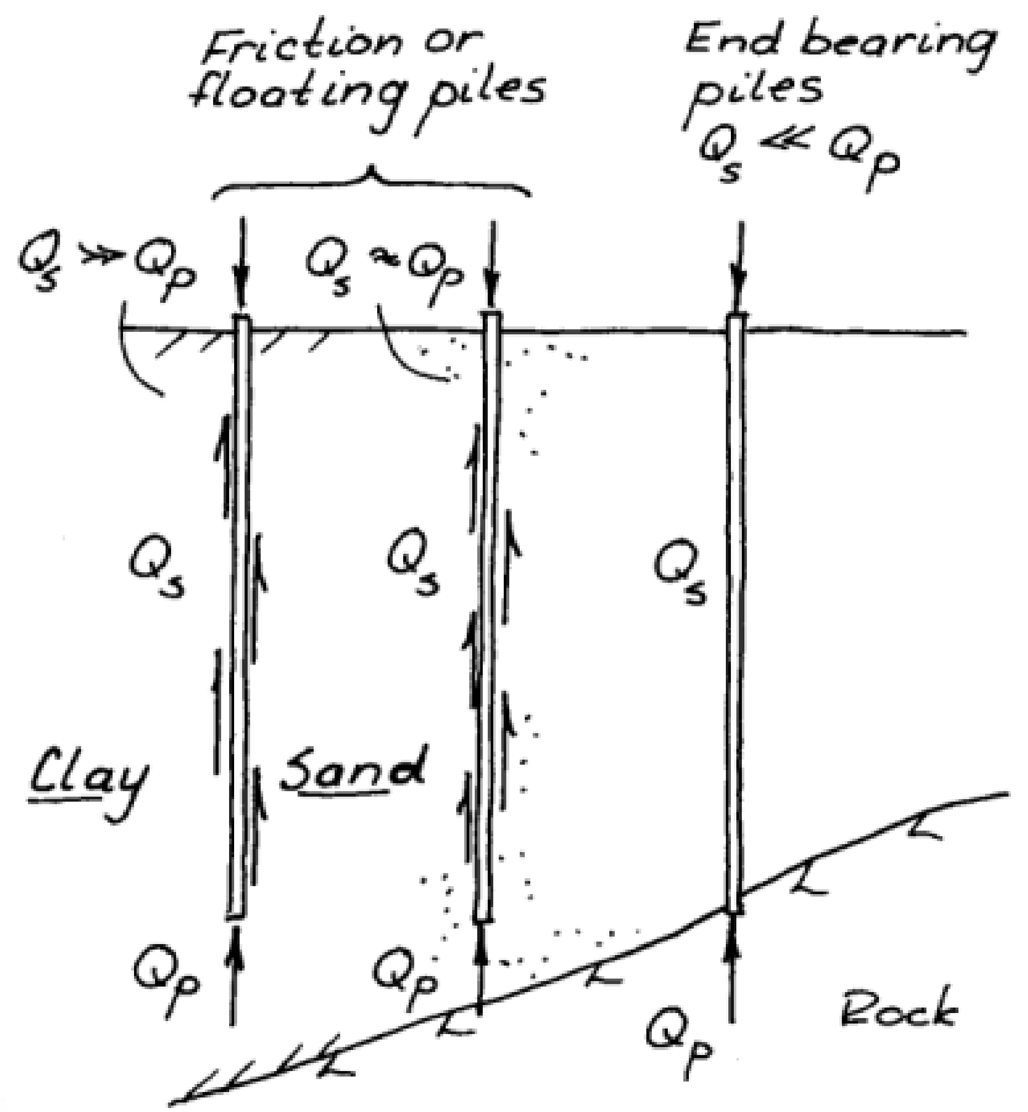
Evaluation Method for Deep Foundations

- Analytic methods, based on soil properties from laboratory and/or in-situ tests
- Load Testing
  - Full-scale static load tests on test piles
  - Dynamic Methods, based on dynamics of pile driving or wave propagation



Concepts to Review

- Shaft and End Bearing Piles
  - Resistance to load
    - Shaft resistance (Qs)
    - Toe resistance (Qt)
  - End-bearing piles, toe resistance predominates
  - Shaft Friction Piles, shaft resistance predominates
  - For tension piles, shaft resistance predominates (toe cannot be in tension)
- Ultimate vs. Allowable Capacity
  - Ultimate capacity is the load required to cause failure, whether by excessive settlement or irreversible movement of the pile relative to the soil
    - For driven piles, one must also consider the resistance to driving, which can be different from the ultimate capacity
  - Allowable capacity is the ultimate capacity divided by a factor of safety (ASD)
  - Important to distinguish between the two



Prediction and Verification

- At what load will the pile fail? (Bearing Capacity)
- How much will pile deflect under service loads? (Settlement)
- The two concepts are not really separated except by custom and code

<u>Prediction</u> on basis of site investigation and laboratory testing

<u>Verification</u> by some method of load testing

Static Load Tests

Figure 9-65. Load test movement monitoring components (FHWA, 2006a).

- The most precise – if not always the most accurate – method of determining the ultimate upward or downward load capacity of a deep foundation
- Static load tests, however, are time consuming and expensive; must be used judiciously
- Object of the test is to develop a load-displacement curve, from which the load capacity can be determined

Dead Load Test

- Considered a reliable static test method
- Slow and expensive
- Dangerous when done unsafely
- No longer commonly used in the U.S.; used where labor costs are lower
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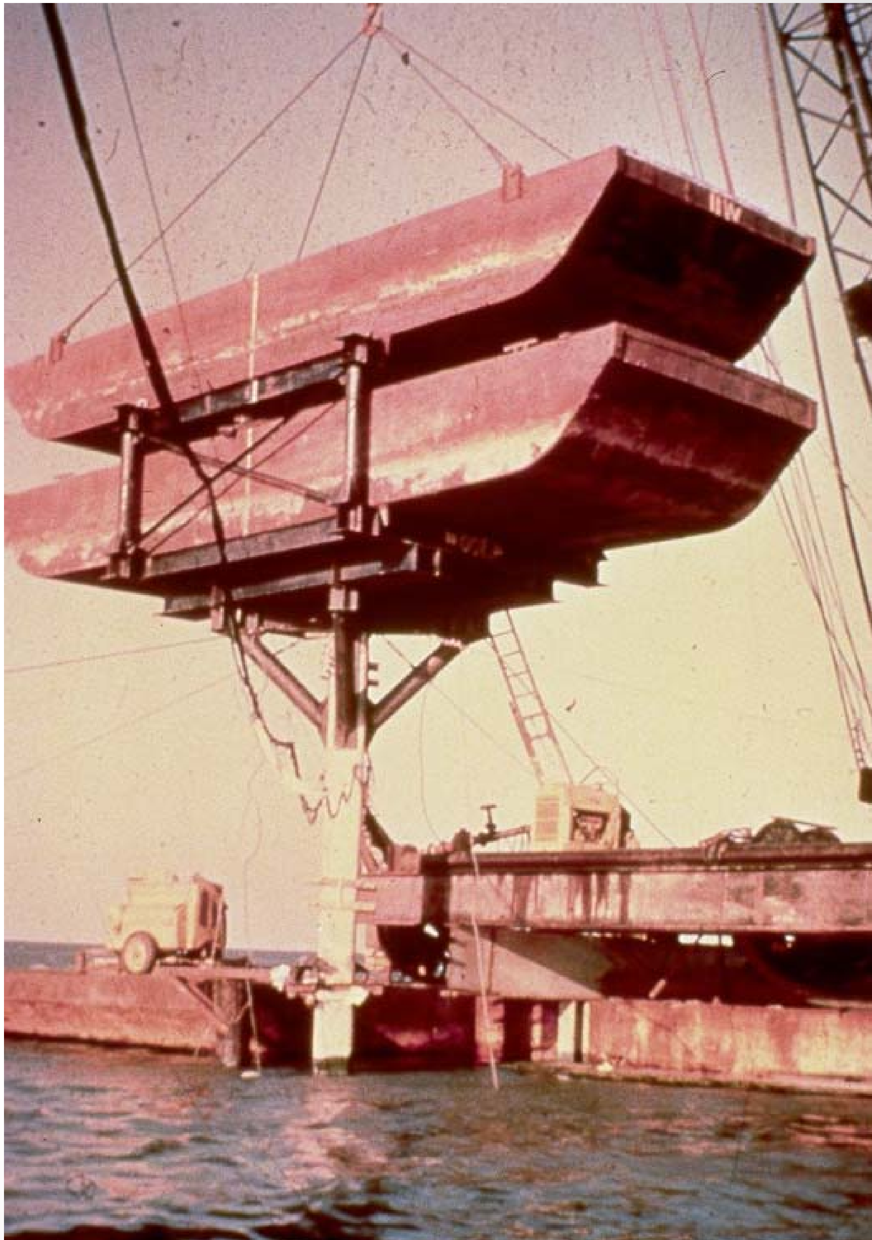


Reaction Pile Systems

- Advantages
  - Can be installed with same equipment as production piles
  - Test can be done on inclination (batter)

Disadvantages

Reaction piles may pull out

If not done properly, reaction pile capacity may result

Flexible system stores energy during tests

Static Load Procedure (ASTM D1143)

- Procedure
  - Set up reaction stand
  - Apply a test load to the pile
  - Record the load-settlement history for each load applied
  - Apply the next load
  - Loads are generally applied in increments of 25, 50, 75, 100, 125, 150, 175 and 200% of proposed design load
- Two categories of tests
  - Controlled stress tests – the most common method
  - Controlled strain tests
- For driven piles, need to delay static load test to allow pile set-up
  - Granular soils – 2 days
  - Cohesive soils – 30 days

Static Load Procedure (ASTM D1143)

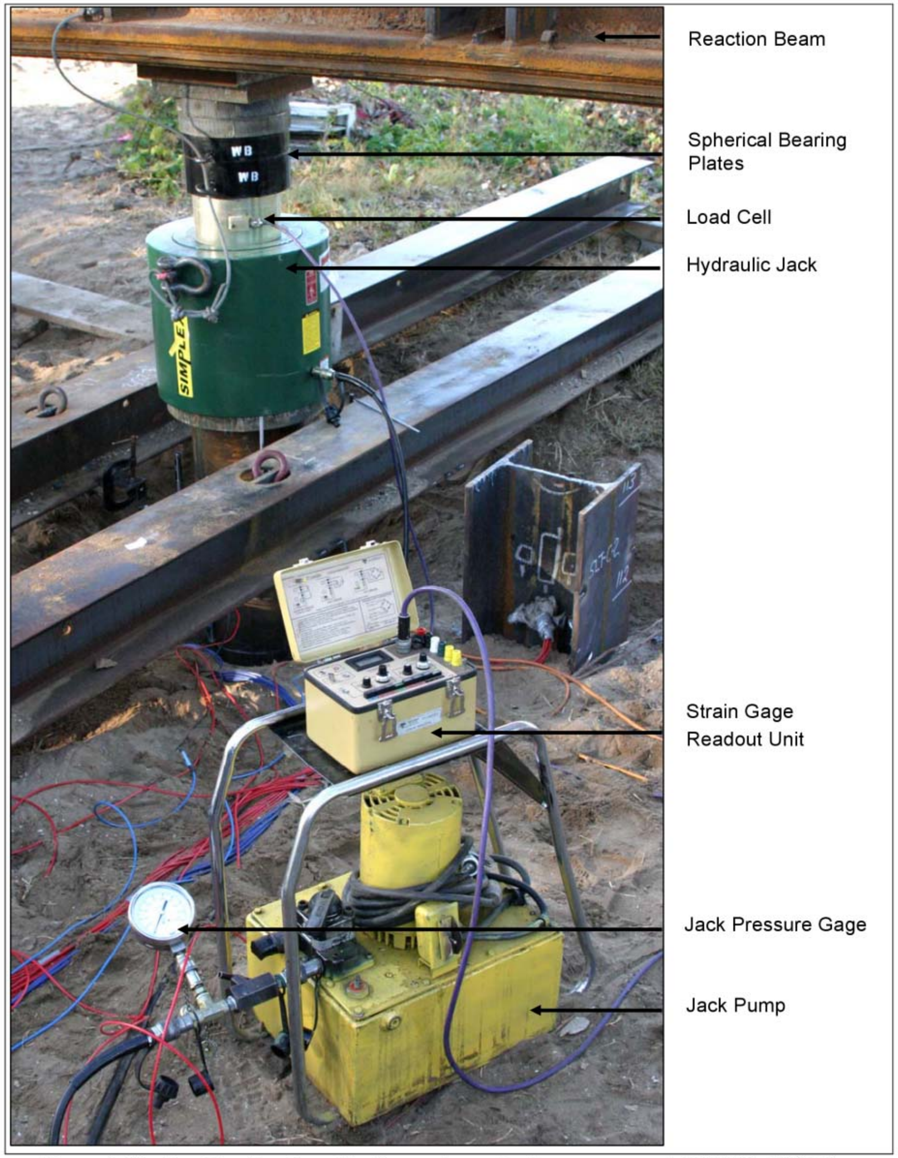


Figure 9-64. Load test load application and monitoring components (FHWA, 2006a).

- Load test increments in time
  - Slow test – maintains load until pile movement is sufficiently small
  - Quick test – each load increment is held for a predetermined length of time, for 2.5 – 15 minutes
    - Generally requires 2-5 hours to complete
    - May be best method for most deep foundations

Static Load Tests: Advantages and Disadvantages

- Advantages
  - Gives reference capacity
  - Relatively slow loading minimises dynamic components
  - Can customise to include creep effects
  - Can be instrumented to yield static resistance distribution & end bearing

Disadvantages

- Time consuming
- Expensive
- Done on specially designated piles
- Often done carelessly or inaccurately

Static Load Test Interpretation

At what point of the measured pile top load vs deflection curve do we define the failure load R_u?

Loading method and curve interpretation can make significant differences in result.

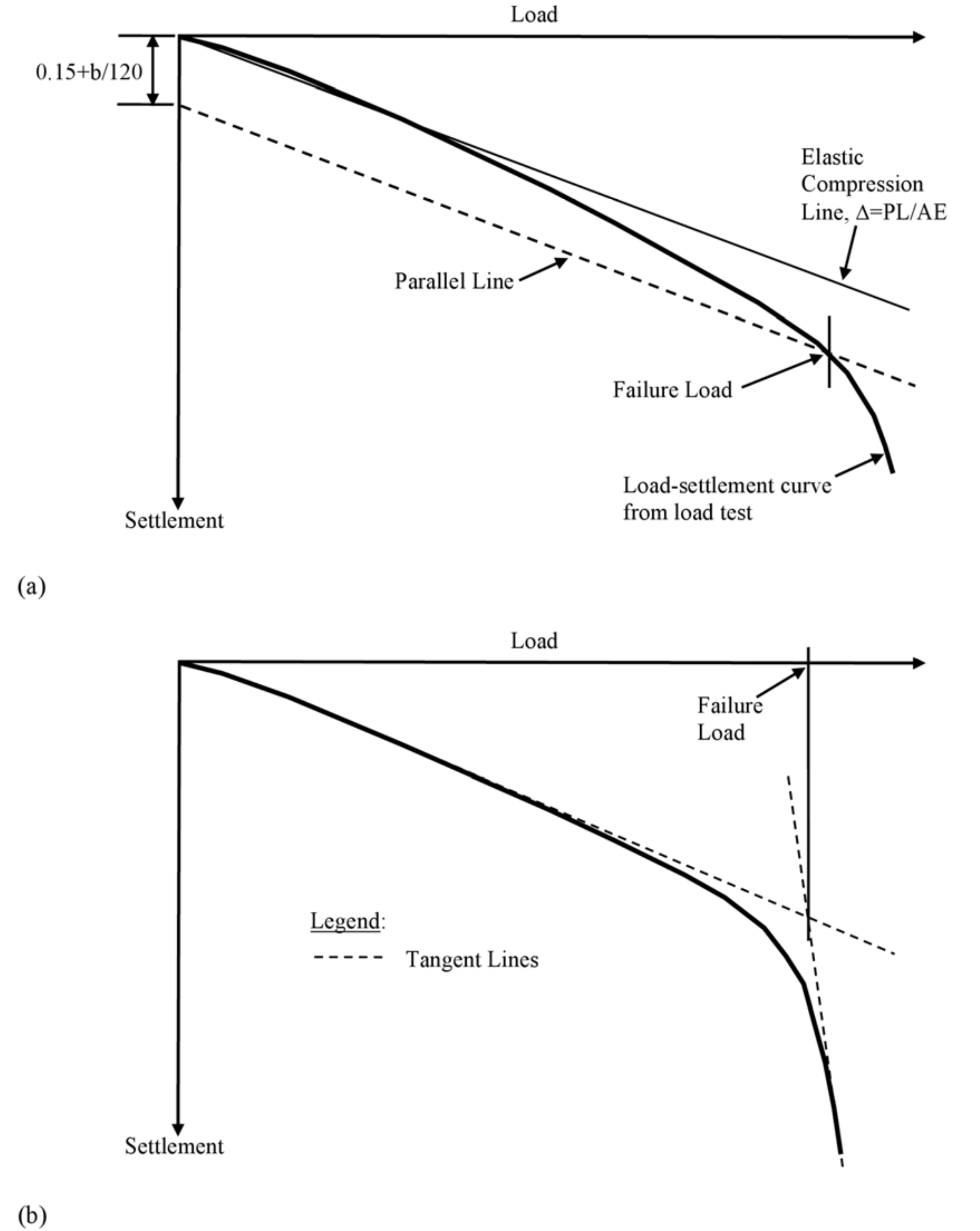


Figure 9-68. Presentation of typical static pile load-movement results, (a) Davisson's method, (b) Double-tangent method.

Davisson's Method

9.15.7.3 Presentation and Interpretation of Compression Test Results

The results of load tests should be presented in a report conforming to the requirements of ASTM D 1143. A load-movement curve similar to the one shown in Figure 9-68 should be plotted for interpretation of test results.

The literature abounds with different methods of defining the failure load from static load tests. Methods of interpretation based on maximum allowable gross movements, which do not take into account the elastic deformation of the pile shaft, are not recommended. These methods overestimate the allowable capacities of short piles and underestimate the allowable capacities of long piles. Methods that account for elastic deformation and are based on a specified failure criterion provide a better understanding of pile performance and provide more accurate results.

AASHTO (2002) and FHWA (1992c) recommend pile compression test results be evaluated by using an offset limit method as proposed by Davisson (1972). The "double-tangent" is more commonly used for drilled shafts. These methods are shown in Figure 9-68 and are discussed in the following sections.

9.15.7.4 Plotting the Failure Criteria

Figure 9-68a shows the load-movement curve from a typical pile load test. To facilitate the interpretation of the test results, the scales for the loads and movements are selected so that the line representing the elastic deformation Δ of the pile is inclined at an angle of about 20° from the load axis. The elastic deformation Δ is computed from:

$$\Delta = \frac{QL}{AE} \qquad 9\text{-}51$$

Where:
- Δ = elastic deformation in inches (mm)
- Q = test load in kips (kN)
- L = pile length in inches (mm)
- A = cross sectional area of the pile in in^2 (m^2)
- E = modulus of elasticity of the pile material in ksi (kPa)

9.15.7.5 Determination of the Ultimate (Failure) Load

For pile diameters less than 24 in (610 mm), the ultimate or failure load Q_f of a pile is that load which produces a movement of the pile head equal to:

In US Units

$$s_f = \Delta + \left(0.15 + \frac{b}{120}\right) \qquad 9\text{-}52$$

where:
- s_f = settlement at failure in inches
- b = pile diameter or width in inches
- Δ = elastic deformation of total pile length in inches

A failure criterion line parallel to the elastic deformation line is plotted as shown in Figure 9-68a. The point at which the observed load-movement curve intersects the failure criterion is by definition the failure load. If the load-movement curve does not intersect the failure criterion line, the pile has an ultimate capacity in excess of the maximum applied test load.

For pile diameters greater than 24 in (610 mm), additional pile toe movement is necessary to develop the toe resistance. For pile diameters greater than 24 in (610 mm), the failure load can be defined as the load that produces at movement at the pile head equal to:

In US Units

$$s_f = \Delta + \left(\frac{b}{30}\right) \qquad 9\text{-}53$$

For drilled shafts, the failure load is commonly determined based on the "double-tangent" method shown in Figure 9-68b. Alternatively, the failure load is often defined as the test load corresponding to 5% of the shaft diameter because such a movement represents a large movement given that the drilled shafts are often much larger in diameter than driven piles.

Example of Davisson's Method

An axial compression static load test has been performed and the results must be interpreted to determine if the pile has an ultimate capacity in excess of the required ultimate capacity. The load - movement curve from the static load on a 356 mm square prestressed concrete pile is presented on the following page. The pile has a cross sectional area, A, of 0.127 m^2 and a length, L, of 24 m. The concrete compression strength, f'_c is 34.5 MPa. The pile has a required ultimate pile capacity of 2200 kN.

Recommended Procedure:

First determine, the elastic modulus, E, of the pile from the concrete compressive strength using $E = 4700 \sqrt{f'_c}$ where f'_c must be in MPa.

$$E = 4700 \sqrt{34.5} = 27606 \text{ MPa}$$

Next, calculate and plot the elastic deformation line using zero and any other load. However, for consistency between solutions and ease in plotting, calculate the elastic deformation using a load of 2500 kN from $\Delta = QL / AE$. Make sure the units for the terms in this equation are as required in the equation description provided in Section 19.7.4.

$\Delta = QL / AE = [2500 \text{ kN } (24 \text{ m})(1000 \text{ mm / m})]/[0.127 \text{ m}^2 (27606000 \text{ kPa})] = 17.1 \text{ mm}$

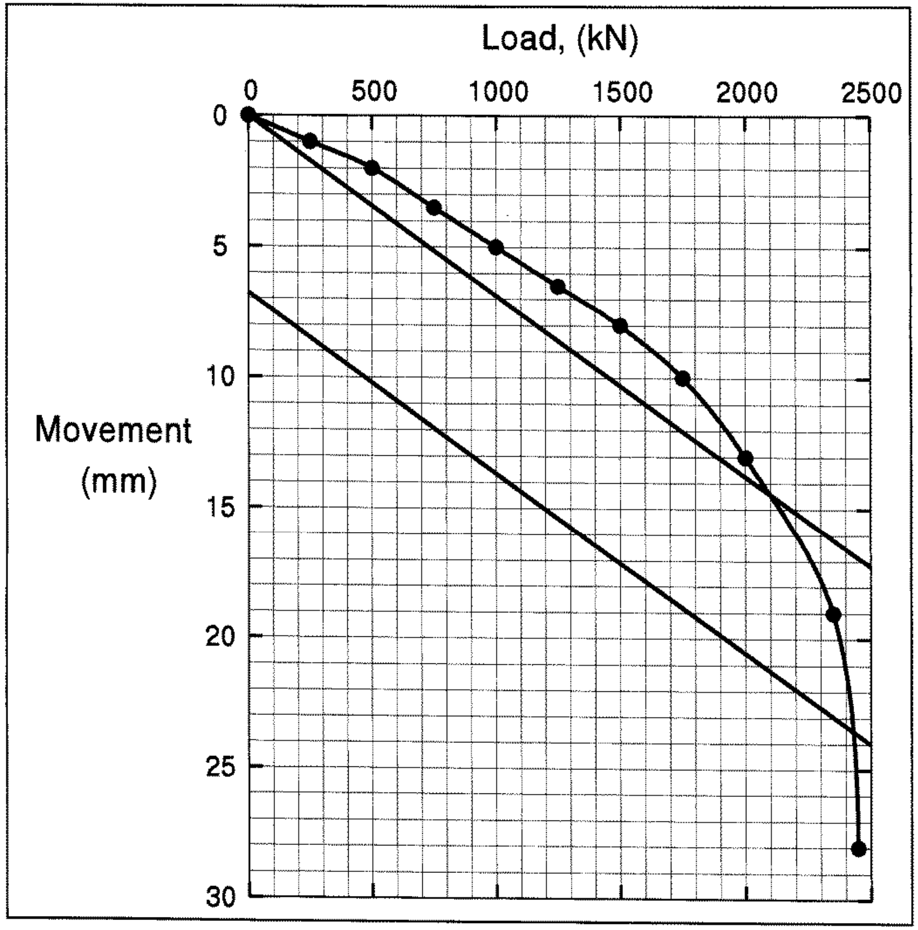
Then calculate the failure criterion line for the 356 mm pile from $s_f = \Delta + (4.0 + 0.008b)$ as described in Section 19.7.5. Remember at zero load, the failure criterion line will start at a movement equal to (4.0 + 0.008b) and at 2500 kN, the failure criterion line will be equal to a movement of $s_f = \Delta + (4.0 + 0.008b)$.

At 0 kN, $s_f = (4.0 \text{ mm} + 0.008b) = 6.8 \text{ mm}$.

At 2500 kN, $s_f = \Delta + (4.0 \text{ mm} + 0.008b) = 17.2 + (4.0 \text{ mm} + 0.008(356)) = 23.9 \text{ mm}$

Last, plot the failure criterion line on the load-movement curve and determine whether the failure load is greater than the required ultimate pile capacity of 2200 kN.

Based on the attached plot, the failure load is 2425 kN which is greater than the required ultimate capacity of 2200 kN.



1.5 Uplift Test

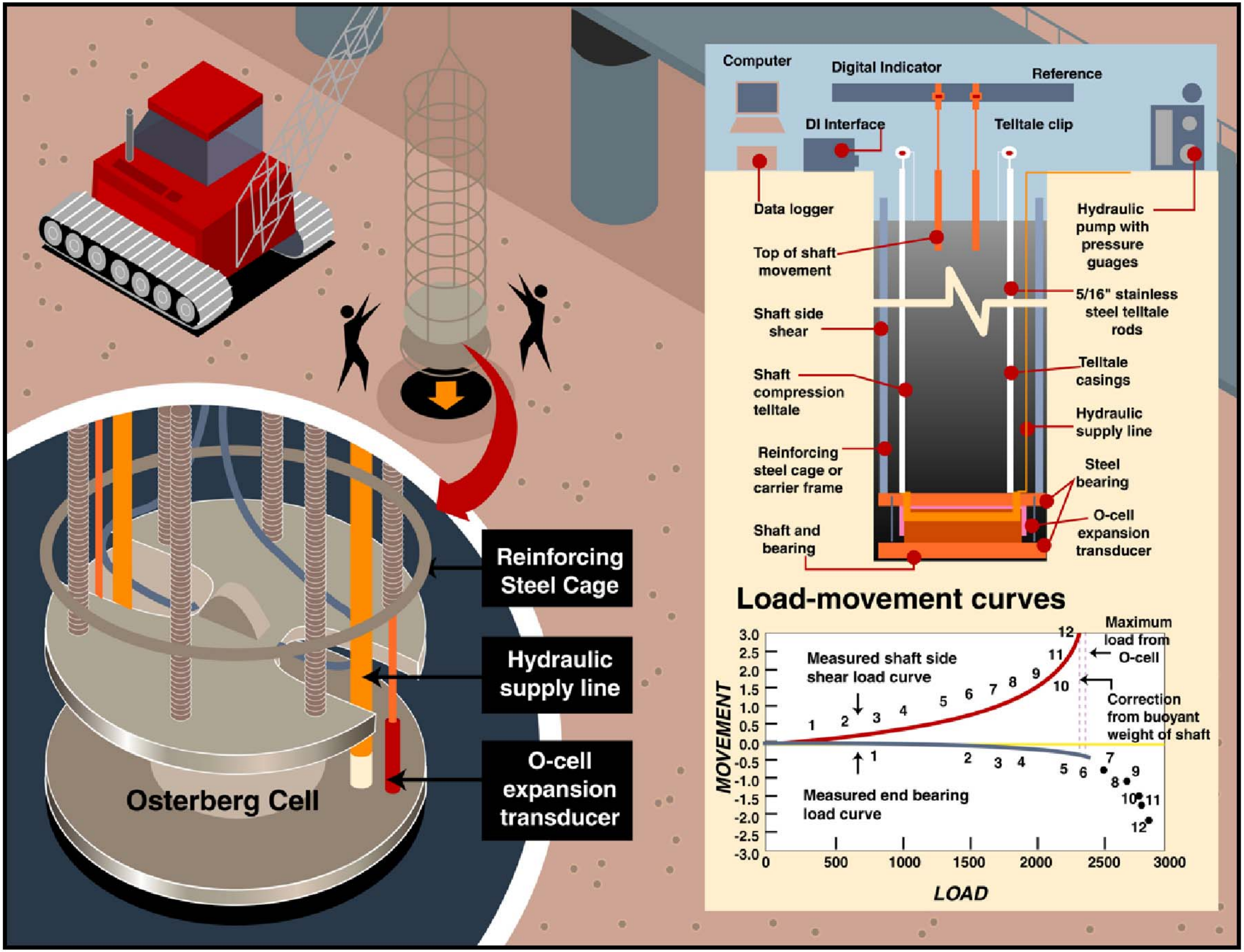
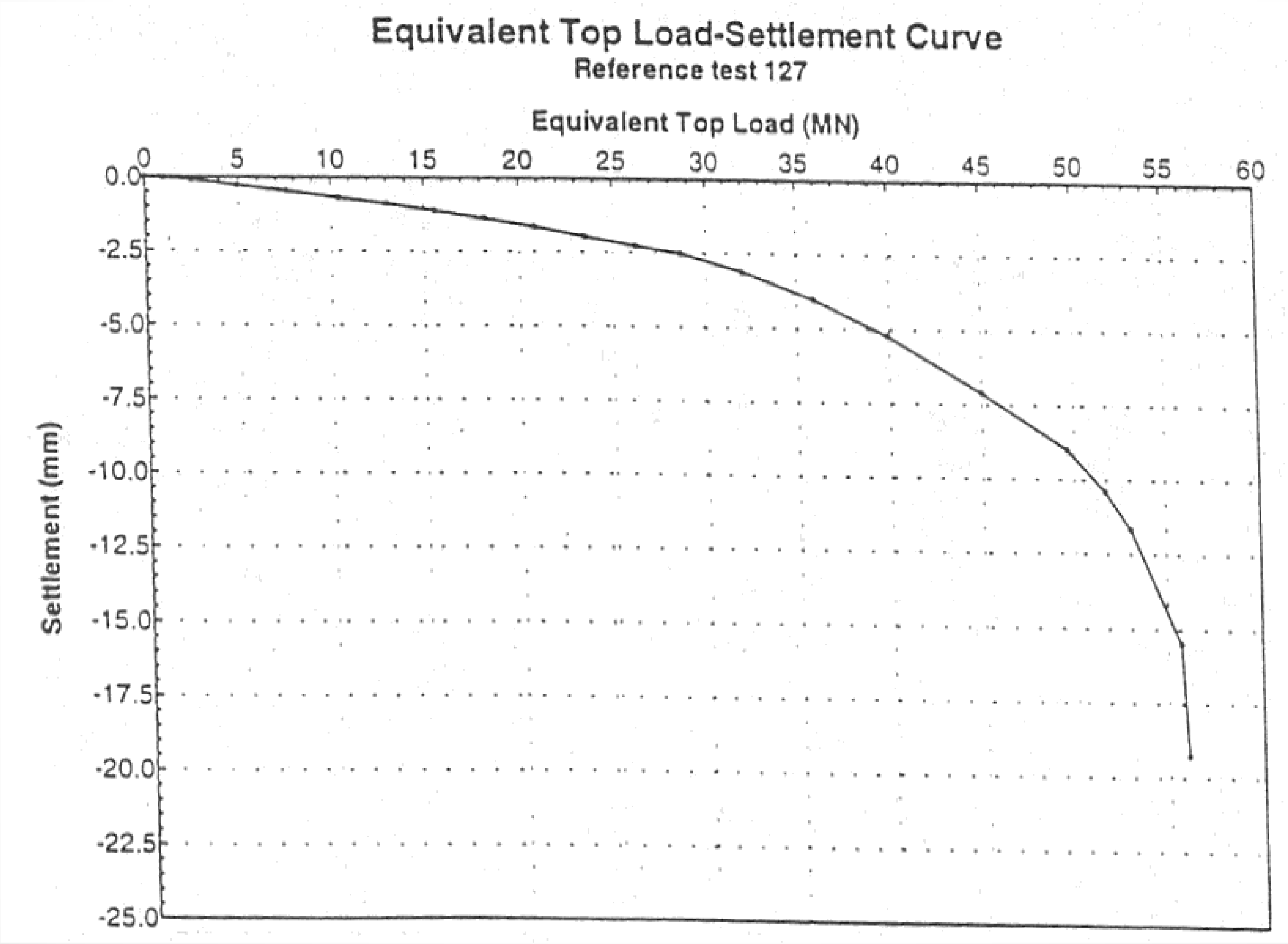


Figure 9-73. Some details of the O-Cell® test (after www.bridgebuildermagazine.com).

Equivalent Curve from Osterberg Data



Osterberg Test
Advantages and Disadvantages

- Shaft is loaded upward rather than in downward direction
- Tensile vertical strains near toe will cause cracking in soil
- Maximum movement is at the pile toe rather than pile top
- Only for specially prepared piles
-

- No reaction load needed
- Requires jack load only half of test load

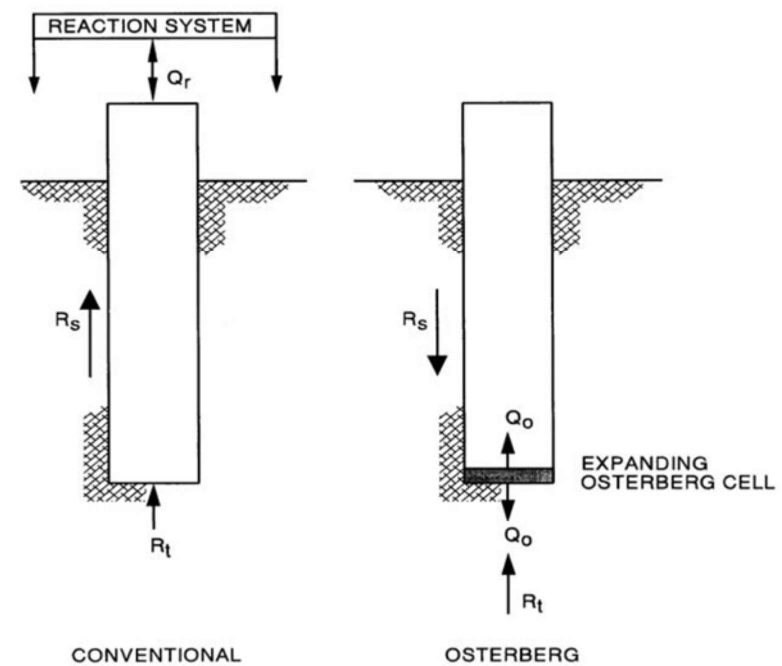


Figure 9-72. Comparison of reaction mechanism between Osterberg Cell® and Static test.

Weaknesses in Static Load Capacity Approach

- Uncertainties in soil-pile interface
- Multiple interpretations of load-deflection curve
  - Pile failure and plunging failure not the same
- Existence of downdrag in piles
- Pile capacity must, in the end, be settlement based

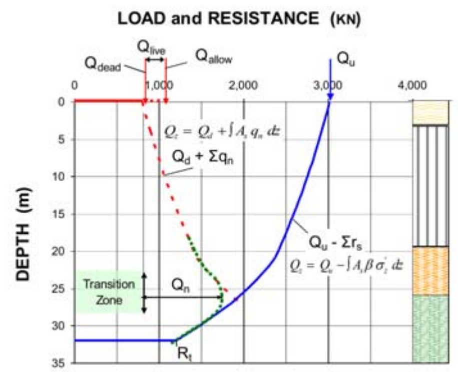


Fig. 7.6a Load-Transfer and Resistance Curves

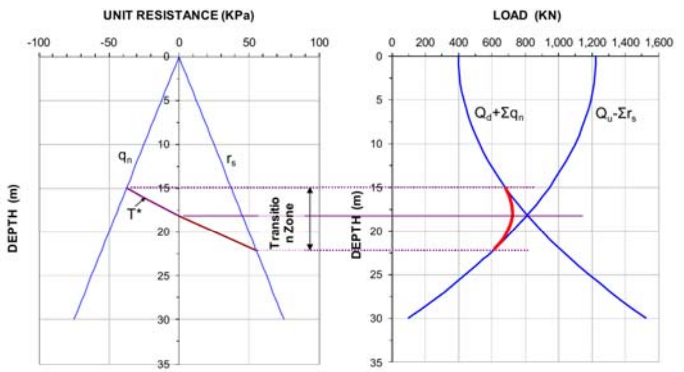


Fig. 7.6b The Principles of Transition from Unit Negative Skin Friction to Unit Positive Shaft Resistance and from Increasing Load to Decreasing Load Distribution (*not the same example as in 7.6a*)

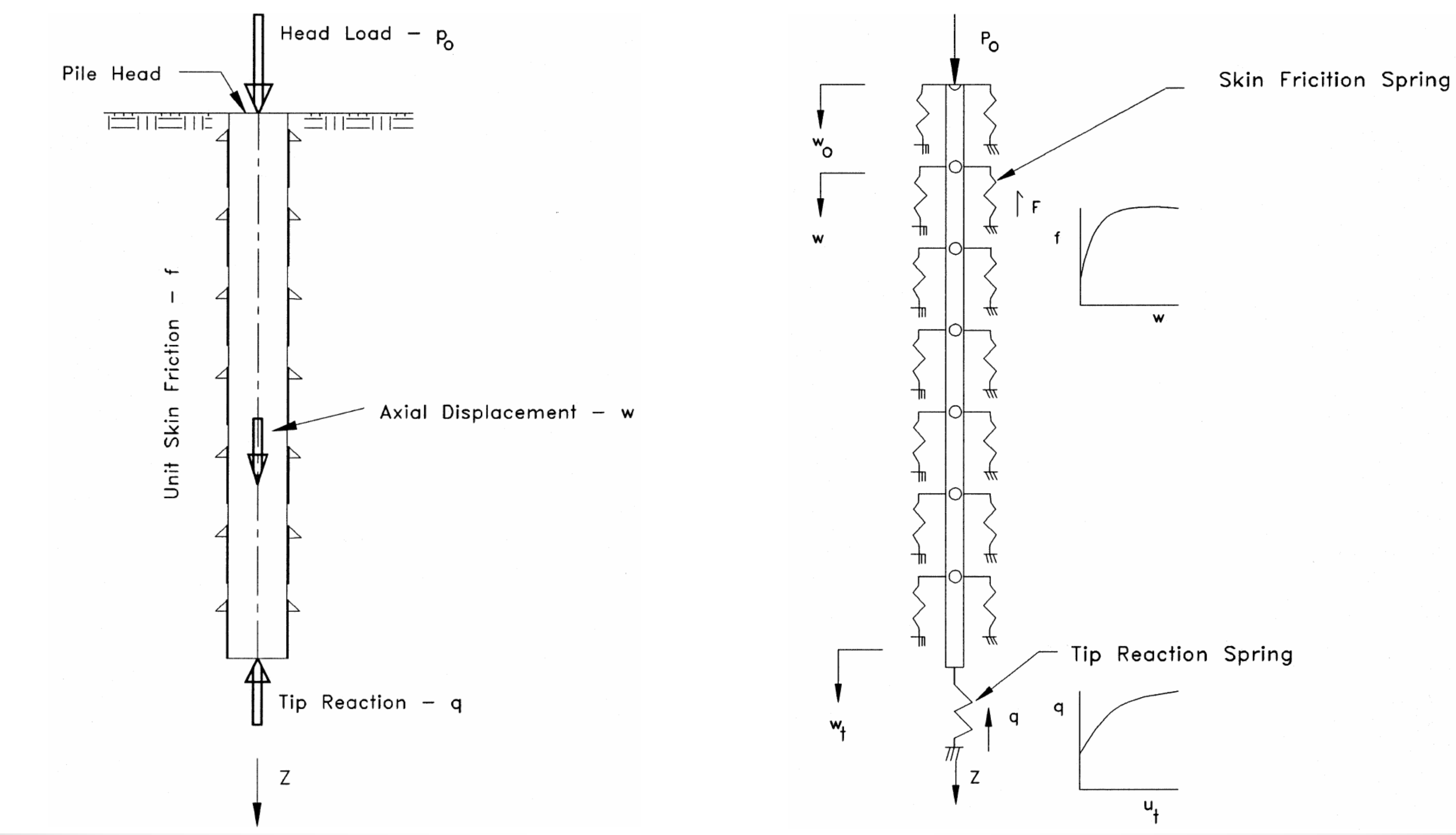
Settlement

- Pile axial capacity is developed by "mobilizing" the shaft and toe resistance
- Without pile deflection, there is no axial resistance to loading
- Pile capacity thus cannot be considered without considering pile settlement
- As with shallow foundations, settlement must ultimately be the main analysis

In the case of "critical" structures, settlement analysis will be performed using a "t-z" method computer program

- Example of one is in the wave equation analysis routine at vulcanhammer.info
- Another is the ALP program, which we will use

t-z Method of Axial Load and Settlement Analysis



Settlement Example

- Given
  - 16” Square Concrete Pile
  - 60’ long
  - Water table 40’ below ground surface
  - Driven into medium SP sands
- Find
  - Load-settlement curve using TAMWAVE software
  - Ultimate Capacity Using Davisson’s Method
- Solution
  - Program returns 120 pcf unit weight, 32 degree internal friction angle (no cohesion), N_{60} = 20
  - We will accept these values, but have the option to change them

Settlement Example

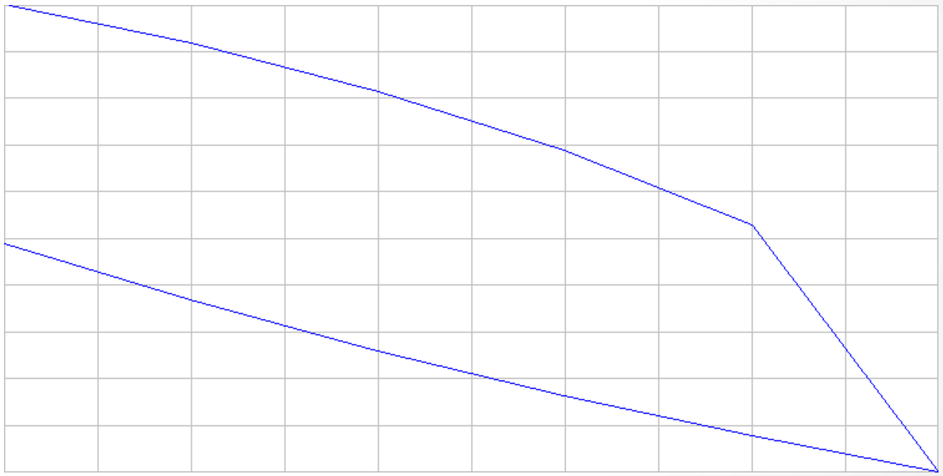
Pile Ultimate Capacity Analysis Results

Pile Data	
Pile Designation	16 In. Square
Pile Material	Concrete
Penetration of Pile into the Soil, ft.	60
Basic "diameter" or size of the pile, ft.	1.3333333333333
Cross-sectional Area of the Pile, ft^2	1.778
Pile Toe Area, ft^2	1.778
Perimeter of the Pile, ft.	5.333
Soil Data	
Type of Soil	SP
Specific Gravity of Solids	2.65
Void Ratio	0.51
Dry Unit Weight, pcf	109.5
Saturated Unit Weight, pcf	130.5
Soil Internal Friction Angle phi, degrees	32
Cohesion c, psf	0
SPT N_{60}, blows/foot	20
CPT q_c, psf	211,600
Distance of Water Table from Soil Surface, ft.	40
Penetration of Pile into Water Table, ft.	20
Active Earth Pressure Coefficient (K_{min})	0.701
Frictional Angle Between Pile and Soil delta, degrees	27.9
Minimum Value for Beta	0.372

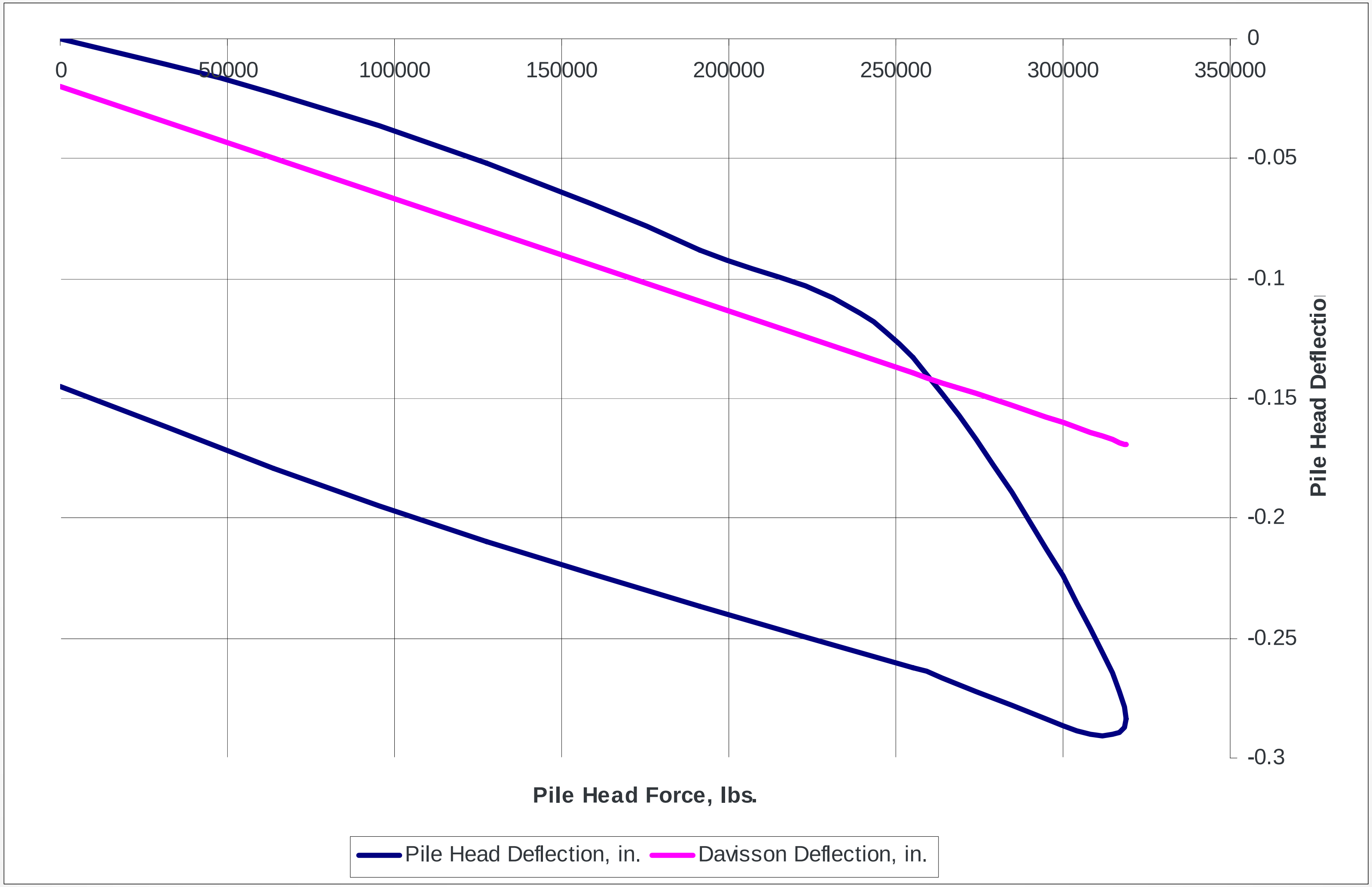
Pile Toe Results	
Effective Stress at Pile Toe, ksf	5.741
N_q	35.4
Relative Density at Pile Toe, Percent	45
SPT $(N_1)_{60}$ at pile toe, blows/foot	12
Unit Toe Resistance q_p, ksf	203.4
Shear Modulus at Pile Toe, ksf	543.3
Toe Spring Constant Depth Factor	1.376
Toe Spring Constant, kips/ft	3,392.2
Pile Toe Quake, in.	1.279
Poisson's Ratio at Pile Toe	0.310
Toe Damping, kips-sec/ft	18.8
Toe Smith-Type Damping Constant, sec/ft	0.052
Total Static Toe Resistance Q_p, kips	361.64
Pile Toe Plugged?	No
Final Results	
Total Shaft Friction Q_s, kips	276.16
Ultimate Axial Capacity of Pile, kips	637.80
Pile Setup Factor	1.0
Total Pile Soil Resistance to Driving (SRD), kips	637.80

Settlement Example

Load Step	Force at Pile Head, kips	Pile Head Deflection, in.	Number of Plastic Shaft Springs
0	0	0	0
1	63.8	0.023	12
2	127.6	0.052	24
3	191.3	0.088	35
4	255.1	0.133	46
5	318.9	0.284	60
6	255.1	0.262	4
7	191.3	0.237	12
8	127.6	0.21	18
9	63.8	0.179	24
10	0	0.145	30

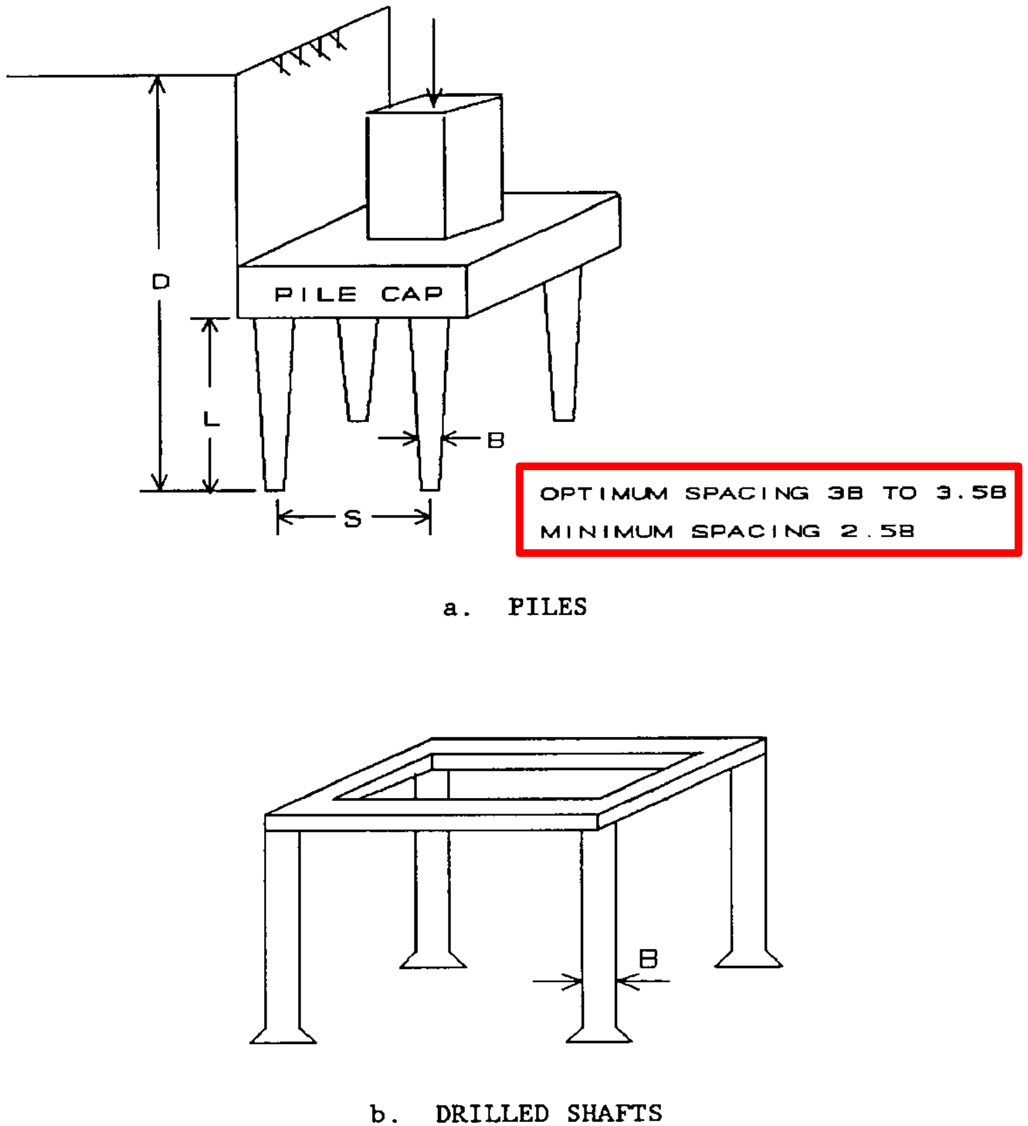


Settlement Example



Davisson Capacity about 260 kips, at intersection of lines

Group Effects



- Piles are generally used in groups; drilled shafts are less frequently so
- Group capacity can be less than the sum of the individual capacities of the piles, depending upon a number of factors
- Group settlements can also be driven by different considerations than settlements of single piles

Stress Zones in Supporting Soils

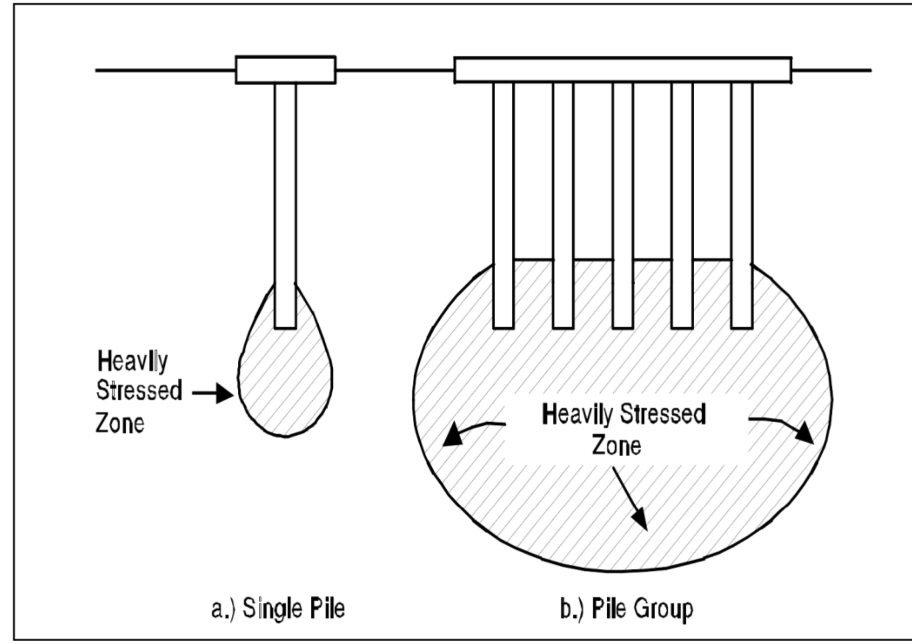


Figure 9-31. Stress zone from single pile and pile group (after Tomlinson, 1994).

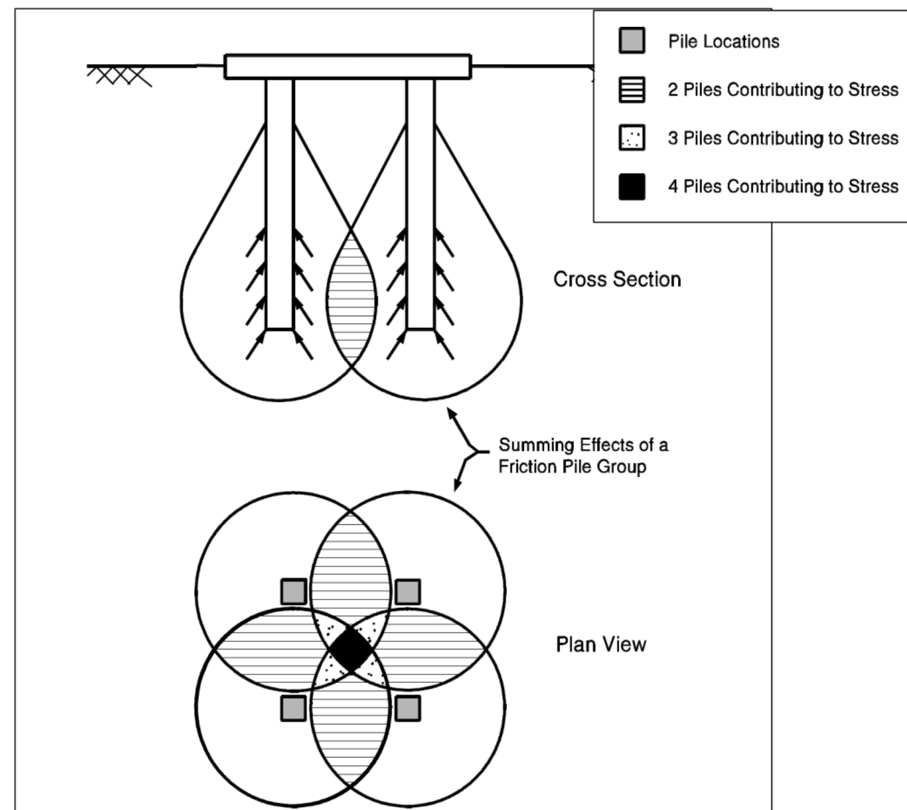


Figure 9-32. Overlap of stress zones for friction pile group (after Bowles, 1996).

9.6 DESIGN OF PILE GROUPS

The previous sections of this chapter dealt with design procedures for single piles. However piles for almost all highway structures are installed in groups due to the heavy foundation loads. This section of the chapter will address the foundation design procedures for evaluating the axial compression capacity of pile groups as well as the settlement of pile groups under axial compression loads. The axial compression capacity and settlement of pile groups are interrelated and are therefore presented in sequence.

The efficiency of a pile group in supporting the foundation load is defined as the ratio of the ultimate capacity of the group to the sum of the ultimate capacities of the individual piles comprising the group. This may be expressed in equation form as:

$$\eta_g = \frac{Q_{ug}}{nQ_u} \qquad 9\text{-}17$$

where:
- η_g = pile group efficiency
- Q_{ug} = ultimate capacity of the pile group
- n = number of piles in the pile group
- Q_u = ultimate capacity of each individual pile in the pile group

If piles are driven into compressible cohesive soil or into dense cohesionless material underlain by compressible soil, then the ultimate axial compression capacity of a pile group may be less than that of the sum of the ultimate axial compression capacities of the individual piles. In this case, the pile group has a group efficiency of less than 1. In cohesionless soils, the ultimate axial compression capacity of a pile group is generally greater than the sum of the ultimate axial compression capacities of the individual piles comprising the group. In this case, the pile group has a group efficiency greater than 1.

The settlement of a pile group is likely to be many times greater than the settlement of an individual pile carrying the same per pile load as each pile in the group. Figure 9-31(a) illustrates that for a single pile, only a relatively small zone of soil around and below the pile toe is subjected to vertical stress. Figure 9-31(b) illustrates that for a pile group, a much larger zone of soil around and below the pile group is stressed. The settlement of the pile group may be large depending on the compressibility of the soils within the stressed zone.

The soil supporting a pile group is also subject to overlapping stress zones from individual piles in the group. The overlapping effect of stress zones for a pile group supported by shaft resistance is illustrated in Figure 9-32.

9.6.1 Axial Compression Capacity of Pile Groups

9.6.1.1 Cohesionless Soils

In cohesionless soils, the ultimate group capacity of driven piles with a center to center spacing of less than 3 pile diameters is greater than the sum of the ultimate capacity of the individual piles. The greater group capacity is due to the overlap of individual soil compaction zones around each pile, which increases the shaft resistance due to soil densification. Piles in groups at center to center spacings greater than three times the average pile diameter generally act as individual piles.

Design recommendations for estimating group capacity for driven piles in cohesionless soil are as follows:

1. The ultimate group capacity for driven piles in cohesionless soils not underlain by a weak deposit may be taken as the sum of the individual ultimate pile capacities, provided jetting or predrilling was not used in the pile installation process. Jetting or predrilling can result in group efficiencies less than 1. Therefore, jetting or predrilling should be avoided whenever possible or controlled by detailed specifications when necessary.

1. If a pile group founded in a firm bearing stratum of limited thickness is underlain by a weak deposit, then the ultimate group capacity is the smaller value of either the sum of the ultimate capacities of the individual piles, or the group capacity against block failure of an equivalent pier, consisting of the pile group and enclosed soil mass punching through the firm stratum into the underlying weak soil. From a practical standpoint, block failure in cohesionless soils can only occur when the center to center spacing of the piles is less than 2 pile diameters, which is less than the minimum center to center spacing of 2.5 diameters allowed by the AASHTO code (2002). The method shown for cohesive soils presented in the Section 9.6.1.3 may be used to evaluate the possibility of a block failure.

3. Piles in groups should not be installed at center to center spacings less than 3 times the average pile diameter. A minimum center to center spacing of 3 diameters is recommended to optimize group capacity and minimize installation problems.

9.6.1.2 Cohesive Soils

In the absence of negative shaft resistance, the group capacity in cohesive soil is usually governed by the sum of the ultimate capacities of the individual piles, with some reduction due to overlapping zones of shear deformation in the surrounding soil. Negative shaft resistance is described in Section 9.8 and often occurs when soil settlement transfers load to the pile. The AASHTO (2002) code states that the group capacity is influenced by whether or not the pile cap is in firm contact with the ground. If the pile cap is in firm contact with the ground, the soil between the piles and the pile group act as a unit.

The following design recommendations are for estimating ultimate pile group capacity in cohesive soils. The lesser of the ultimate pile group capacity, calculated from Steps 1 to 4, should be used.

1. For pile groups driven in clays with undrained shear strengths of less than 2 ksf (95 kPa) and for the pile cap not in firm contact with the ground, a group efficiency of 0.7 should be used for center to center pile spacings of 3 times the average pile diameter. If the center to center pile spacing is greater than 6 times the average pile diameter, then a group efficiency of 1.0 may be used. Linear interpolation should be used for intermediate center to center pile spacings.

2. For pile groups driven in clays with undrained shear strengths less than 2 ksf (95 kPa) and for the pile cap in firm contact with the ground, a group efficiency of 1.0 may be used.

3. For pile groups driven in clays with undrained shear strength in excess of 2 ksf (95 kPa), a group efficiency of 1.0 may be used regardless of the pile cap - ground contact.

4. Calculate the ultimate pile group capacity against block failure by using the procedure described in Section 9.6.1.3.

5. Piles in groups should not be installed at center to center spacings less than 3 times the average pile diameter and not less than 3 ft (1 m).

It is important to note that the driving of pile groups in cohesive soils can generate large excess pore water pressures. The excess pore water pressures can result in short term group efficiencies on the order of 0.4 to 0.8 for 1 to 2 months after installation. As these excess pore water pressures dissipate, the pile group efficiency will increase. Figure 9-33 presents observations on the dissipation of excess pore water pressure versus time for pile groups driven in cohesive soils.

9.6.1.3 Block Failure of Pile Groups

Block failure of pile groups is generally a design consideration only for pile groups in soft cohesive soils or in cohesionless soils underlain by a weak cohesive layer. For a pile group in cohesive soil as shown in Figure 9-34, the ultimate capacity of the pile group against a block failure is provided by the following expression:

$$Q_{ug} = 2D\,(B + Z)\,c_{u1} + B\,Z\,c_{u2}\,N_c \qquad \text{9-18}$$

where:
- Q_{ug} = ultimate group capacity against block failure
- D = embedded length of piles
- B = width of pile group
- Z = length of pile group
- c_{u1} = weighted average of the undrained shear strength over the depth of pile embedment for the cohesive soils along the pile group perimeter
- c_{u2} = average undrained shear strength of the cohesive soils at the base of the pile group to a depth of 2B below pile toe level
- N_c = bearing capacity factor

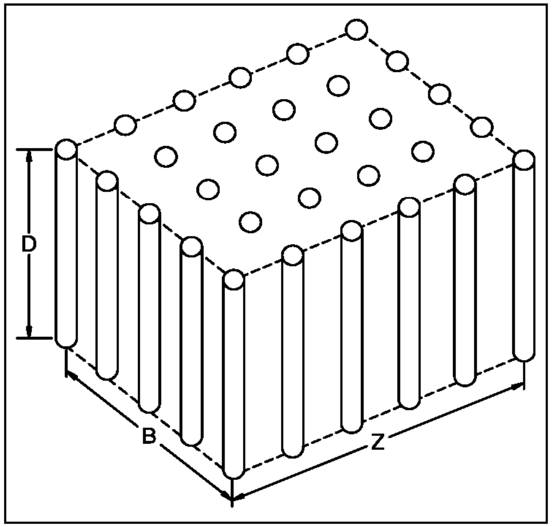


Figure 9-34. Three dimensional pile group configuration (after Tomlinson, 1994).

If a pile group will experience the full group load shortly after construction, the ultimate group capacity against block failure should be calculated by using the remolded or a reduced shear strength rather than the average undrained shear strength for c_{u1}.

The bearing capacity factor, N_c, for a rectangular pile group is generally 9. However, for pile groups with relatively small pile embedment depths and/or relatively large widths, N_c should be calculated from the following equation where the terms D, B and Z are as shown in Figure 9-34.

$$N_c = 5\left(1 + \frac{D}{5B}\right)\left(1 + \frac{B}{5Z}\right) \leq 9 \qquad \text{9-19}$$

In the evaluation of possible block failure of pile groups in cohesionless soils underlain by a weak cohesive deposit, the weighted average unit shaft resistance for the cohesionless soils should be substituted for c_{u1} in calculating the ultimate group capacity. The pile group base strength determined from the second part of the ultimate group capacity equation should be calculated by using the strength of the underlying weaker layer.

9.6.2 Settlement of Pile Groups

Pile groups supported in and underlain by cohesionless soils will produce only elastic or immeidate settlements. This means that the settlements will occur almost immediately as the pile group is loaded. Pile groups supported in and underlain by cohesive soils may produce both elastic settlements that will occur almost immediately and consolidation settlements that will occur over a period of time. In highly over-consolidated clays, the majority of the foundation settlement will occur almost immediately. Consolidation settlements will generally be the major source of foundation settlement in normally consolidated clays.

Methods for estimating settlement of pile groups are provided in the following sections. Methods for estimating single pile settlements are not provided in this document because piles are usually installed in groups.

9.6.2.1 Elastic Compression of Piles

The methods for computing pile group settlement discussed in the following sections consider soil settlements only and do not include the settlement caused by elastic compression of pile material due to the imposed axial load. Therefore, the elastic compression should also be computed and added to the group settlement estimates of soil settlement to obtain the total settlement. The elastic compression can be computed by the following expression:

$$\Delta = \frac{Q_a L}{A E} \qquad \text{9-20}$$

where:
- Δ = elastic compression of pile material in inches (mm)
- Q_a = design axial load in pile in kips (kN)
- L = length of pile in inches (mm)
- A = pile cross sectional area in in^2 (mm^2)
- E = modulus of elasticity of pile material in ksi (kPa)

The modulus of elasticity for steel piles is 30,000 ksi (207,000 MPa). For concrete piles, the modulus of elasticity varies with concrete compressive strength and is generally on the order of 4,000 psi (27,800 MPa). The elastic compression of short piles is relatively small and can often be neglected in design.

9.6.2.2 Settlement of Pile Groups in Cohesionless Soils

Meyerhof (1976) recommended the settlement of a pile group in a homogeneous sand deposit not underlain by a compressible soil be conservatively estimated by the following expressions in U.S. units:

$$s = \frac{4\, p_f \sqrt{B}\, I_f}{\overline{N'}} \qquad \text{For silty sand, use: } s = \frac{8\, p_f \sqrt{B}\, I_f}{\overline{N'}} \qquad \text{9-21}$$

where:
- s = estimated total settlement in inches
- p_f = design foundation pressure in ksf = group design load divided by group area
- B = width of pile group in ft
- $\overline{N'}$ = average corrected SPT $N1_{60}$ value within a depth B below pile toe
- I_f = influence factor for group embedment $= 1 - [D / 8B] \geq 0.5$
- D = pile embedment depth in ft

9.6.2.3 Settlement of Pile Groups in Cohesive Soils

Terzaghi and Peck (1967) proposed that pile group settlements could be evaluated using an equivalent footing situated at a depth of D/3 above the pile toe. This concept is illustrated in Figure 9-35. For a pile group consisting of only vertical piles, the equivalent footing has a plan area (B)(Z) that corresponds to the perimeter dimensions of the pile group as shown in Figure 9-34. The pile group load over this plan area is then the bearing pressure transferred to the soil through the equivalent footing. The load is assumed to spread within the frustum of a pyramid of side slopes at 30° and to cause uniform additional vertical pressure at lower levels. The pressure at any level is equal to the load carried by the group divided by the plan area of the base of the frustum at that level. Once the equivalent footing dimensions have been established then the settlement of the pile group can be estimated by using the procedures described in Chapter 8 (Shallow Foundations).

Rather than fixing the equivalent footing at a depth of D/3 above the pile toe for all soil conditions, the depth of the equivalent footing should be adjusted based upon soil stratigraphy and load transfer mechanism to the soil. Figure 9-36 presents the recommended location of the equivalent footing for the following load transfer and soil resistance conditions:

a) toe bearing piles in hard clay or sand underlain by soft clay
b) piles supported by shaft resistance in clay
c) piles supported in shaft resistance in sand underlain by clay
d) piles supported by shaft and toe resistance in layered soil profile

Note that Figures 9-35 and 9-36 assume that the pile group consists only of vertical piles. If a group of piles contains battered piles, then they should be included in the determination of the equivalent footing width only if the stress zones from the battered piles overlap with those from the vertical piles.

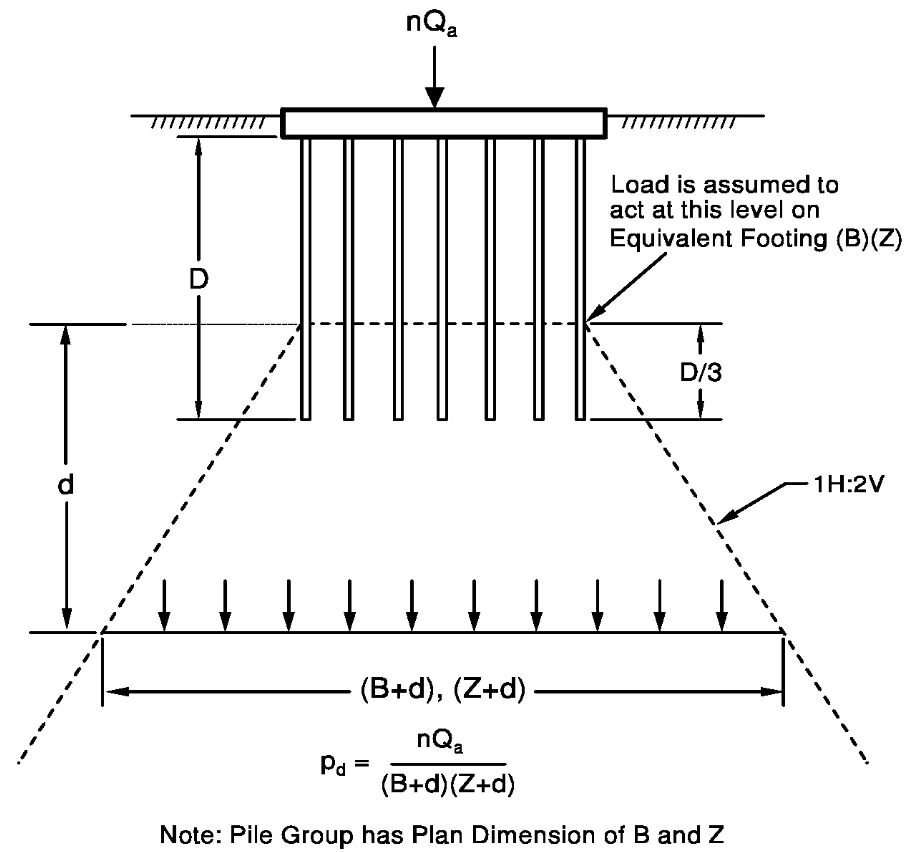


Figure 9-35. Equivalent footing concept (after Duncan and Buchignani, 1976).

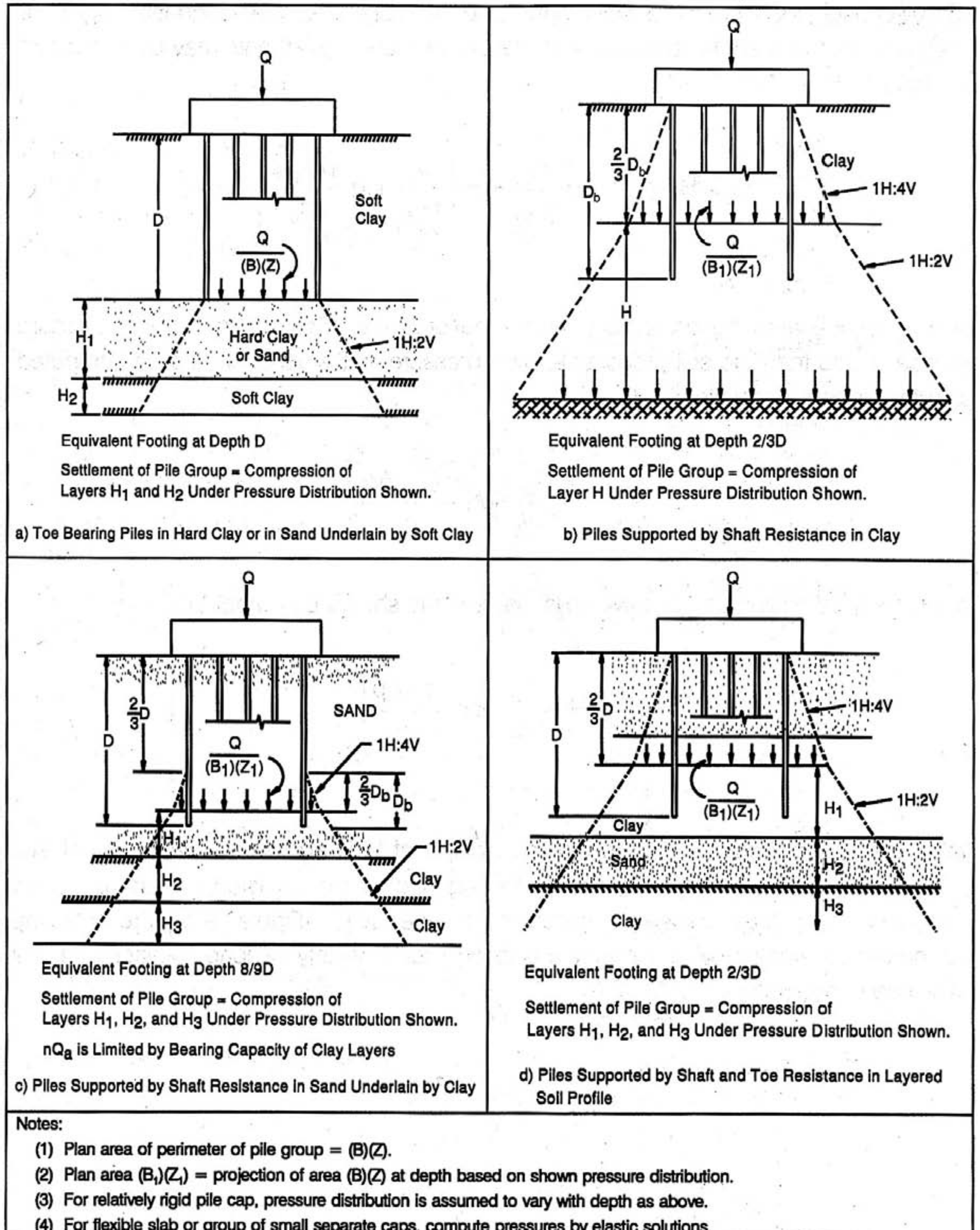


Figure 9-36. Stress distribution below equivalent footing for pile group (FHWA, 2006a).

Group Settlement Example

- Pile Group
  - 12" Square Concrete Piles, 50' long/embedment
  - Pile Cap 12' x 12' (B)
  - Piles arranged in a 5 x 5 arrangement (25 piles total)
  - Total cap load 1250 kips
  - Group driven into SW soils, typical $N1_{60}$ value of 20
- Find
  - Estimated group settlement

- Solution
  - Cap area = 144 sq. ft.
  - Overall cap pressure p_f = 1250/144 = 8.68 psf
  - Influence Factor I_f = 1-(50/(8*12)) = 0.479 (must be raised to 0.5)
  - s = (4)(8.68)(5)1/2(0.5)/20
  - s = 1.94"

$$s = \frac{4\, p_f \sqrt{B}\, I_f}{\overline{N'}} \qquad \text{For silty sand, use: } s = \frac{8\, p_f \sqrt{B}\, I_f}{\overline{N'}} \qquad \text{9-21}$$

where:

s	=	estimated total settlement in inches
p_f	=	design foundation pressure in ksf = group design load divided by group area
B	=	width of pile group in ft
$\overline{N'}$	=	average corrected SPT $N1_{60}$ value within a depth B below pile toe
I_f	=	influence factor for group embedment $= 1 - [D/8B] \geq 0.5$
D	=	pile embedment depth in ft

Questions