CHAPTER 6

GEOTECHNICAL ANALYSES

6.0 INTRODUCTION

As soon as the soils exploration program is complete and the data is available, the geotechnical engineer should be ready to start the geotechnical analyses. Good geotechnical analyses begin with a good understanding of the soil data, profile, and parameters.

The intent of this chapter is to provide general guidance to identify the soil and foundation concerns that need to be evaluated, and the current requirements that the analysis should satisfy. It is presumed the engineer is familiar with all aspects of geotechnical engineering, as they relate to the behavior of highway structures and roadways. The term, "analyses" in this section does not, necessarily, include all the mathematics needed to analyze a certain situation.

The design analyses contained in the Geotechnical Report should be in compliance with current requirements given in this INDOT Geotechnical Manual and Guidelines. The current FHWA and NHI manuals should be consulted for more detailed guidelines. For any other specific requirement not covered in this manual prior approval of the Manager, Office of Geotechnical Services should be obtained.

NOTE: All geotechnical designs for shallow and deep foundation (footings, bridges) and other retaining structures will be done by Load Resistance Factor Design (LRFD) method in accordance with the guide lines given in the following documents:

1. AASHTO LRFD bridge design specification 2017 8th edition, or the latest edition.
2. LRFD for highway structures: FHWA-NHI 05-094 and FHWA –NHI -06-098
3. INDOT Spec-2018 or latest edition.

6.1 SETTLEMENT ANALYSIS

This section addresses only the consolidation settlement in the natural ground, under the embankments. Normal construction practices are usually adequate to preclude excessive post construction consolidation within the embankment.

Consolidation settlement takes place when the weight of the embankment exceeds the previous stress history of the underlying strata. In this case, the soil particles are pressed more closely together. The amount of settlement is a direct measurement of the reduction in the soil voids space.

Soil settlement consists of primary and secondary consolidation. Primary consolidation is the portion of the consolidation curve in which the reduction in void ratio is associated with the dissipation of excess pore water pressure. The pore pressure depends on soil permeability, which is a function of the particle size. Granular materials are sufficiently permeable to dissipate excess pore water pressure as quickly as the embankment load is applied. At the other extreme, thick deposits of wet, high clay content soil may not achieve equilibrium pore water pressure for decades.

Secondary consolidation occurs after full dissipation of excess pore water pressure. Secondary consolidation is a problem with high organic deposits, such as peat. For peat, the total secondary consolidation could be twice as much as the primary consolidation. With mineral soils, the secondary consolidation is not commonly considered a problem. The consolidation characteristics of fine-grained soils are evaluated in the laboratory, on specimens taken from undisturbed soil samples.

If consolidation test data is not available, the primary settlement(s) can be estimated using geotechnical parameters obtained from empirical relationships following are empirical formulas suggested by various researchers to calculate compression index (Cc) values.

Table 6.1 Correlation's for Compression Index *Cc**

Equation	Reference	Region of applicability
$C_c = 0.007(LL - 7)$	Skempton	Remolded clays
$C_c = 0.01w_n$		Chicago clays
$C_c = 1.15(e_o - 0.27)$	Nishida	All clays
$C_c = 0.30(e_o - 0.27)$	Hough	Inorganic cohesive soil: silt, silty clay, clay
$C_c = 0.0115\, w_n$		Organic soils, peat's, organic silt, and clay
$C_c = 0.0046(LL - 9)$		Brazilian clays
$C_c = 0.75(e_o - 0.5)$		Soils with low plasticity
$C_c = 0.208e_o + 0.0083$		Chicago clays
$C_c = 0.156e_o + 0.0107$		All Clays

*After Rendon-Herrero (1980)
Note: e_o = *in situ* void ratio; w_n = *in situ* water content

Swell Index (C_s)

The swell index is appreciably smaller in magnitude than the compression index and can generally be determined from laboratory tests. In most cases,

$$S = H\,\frac{C_c}{1+e_o}\log 10\left(\frac{Po+\Delta P}{Po}\right) \qquad \text{Equation (6.1)}$$

Calculation of settlement:

For normally consolidated soils

$$S = H\,\frac{Cc}{1+eo}\log\frac{Po+\Delta P}{Po}$$

Where Po is the existing pressure on the compressible layer due to soil strata above this layer (lb/ft ^{2}).
ΔP = Increase in pressure on the compressible layer due to construction at top (lb/ft^2).
e_o = initial void ratio

Normally consolidated soil is the soil which has not been subjected to higher pressure than existing total pressure (total pressure at present including any additional pressure due to construction at the surface) any time in the past.
Pc = pre-consolidation pressure is the maximum pressure the compressible layer has been subjected to in the past (lb/ft^2).
For over consolidated soil the settlement may be calculated as given below:

If (Po + ΔP) < Pc

$$S = H\,\frac{Ccr}{1+e_0}\log 10\left(\frac{Po+\Delta P}{Po}\right)$$ Equation (6.2)

If (Po +Δ P) > Pc

$$S = H\,\frac{Ccr}{1+eo}\log\,\frac{Pc}{Po} + H\,\frac{C_c}{1+eo}\log\,\left(\frac{Po+\Delta P}{Pc}\right)$$

Equation (6.3)

C_c = Compression index

C_{cr} =Recompression index

H = Thickness of compressible layer (ft.)

e_o = Initial void ratio

For very soft to soft clays (Q_u between 0.25 to 0.50 tsf), the settlements computed by this method are likely to be reasonably accurate. For medium and stiff clays (Q_u between 0.5 and 2.0 tsf), the actual settlements are likely to range between one-fourth and one-tenth of the computed values.

The analysis of a proposed wick drain should include: design spacing at a specific embankment section based on consolidation test results. The consultant geotechnical engineer shall furnish an estimated coefficient of horizontal consolidation, a plot of percent total estimated settlement vs. time using the optimum wick drain design, the limits from station to station and offset to offset where the proposed wick drains should be installed with any other information needed.

6.2 STABILITY OF PAVEMENT SUBGRADE

Subgrade stability must consider the short-term and long-term behavior of the subgrade. The subgrade should adequately support the heavy equipment during construction, with minimum rutting. The subgrade should also support the roadway during its design life.

In addition to the subgrade requirements in the Standard Specification, there are field conditions, which must be considered during the life of the pavement structure. The stress level at the subgrade, under repeated peak axle load repetitions, must be maintained within the range of elastic response of the subgrade soil. Failure to do so will result in the yielding of the subgrade, resulting in loss of pavement support and pavement failure.

Internal drainage of the pavement system and the subgrade can exert a profound influence on the pavement performance. As the ground water rises toward the subgrade, and particularly within the upper 6 inches of a fine grained soil subgrade, the soil is essentially saturated. The result is load support reduction.

6.3 STABILITY OF SLOPES

Slopes of roadway embankments in fill and cut areas should be stable for efficient functioning of roadways. This section describes types and reasons of slope failure including the methodology to check the stability of slopes.

6.3.1 TYPES OF FAILURE

The principle modes of failure (slip) in soil or rock are; 1) rotation on a curved slip surface approximated by a circular arc; 2) translation along a planar surface whose length is large compared to depth below ground elevation; 3) displacement of a wedge shaped mass along one or more planes of weakness. Other modes include: toppling of rock slides, block slides, lateral spreading, earth and mud flows in clayey and silty soils, and debris flows in coarse grained soils.

A slip circle can be a base circle, toe circle, or a slope circle. A base slip circle develops when there is a significant thickness of weak foundation soil. The base of the failure arc is tangent to the base of the weak layer and the arc will have a significant portion of its length in the weak soil. A toe slip circle develops in the embankment and intersects the ground surface at the toe. A slope circle develops within the embankment and intersects the slope face. Sloughing of the slope due to erosion is an example of a slope slip circle.

A planar failure is more commonly associated with the shear plane following a thin zone of weakness, and is seldom far below the base of the embankment or toe of slope. The failure plane may develop at the soil/shale contact, with seepage on the shale surface. The planar failure may also develop at the base of an embankment. This could happen when an organic layer and vegetative cover have been inadequately processed during construction, resulting in a built-in failure plane.

Block movements are more common to cut sections through relatively competent soils; such as a weathered glacial till. The movements take place along secondary structural cracks and joints. Residual soils may also fall into this group, with the plane of movement taking place along relic joints and bedding planes.

6.3.2 REASONS FOR FAILURE

Slope failure takes place when the driving forces exceed the resisting forces. The force imbalance may be caused by one or more of the following situations.

6.3.2.1 EMBANKMENT (FILL) SLOPE

Slope profile changes that add driving weight at the top, or decreases in the resisting forces at the base. Examples would be the steepening of the slope or undercutting of the toe.

Vibrations induced by earthquakes, blasting, or pile driving. Depending on their frequency and intensity, induced dynamic forces could cause either liquefaction or densification of loose sand, silt, and loess below the ground water surface. Dynamic forces could cause the collapse of sensitive clays, thereby, resulting in increased pore pressures.

Overstressing of the foundation soil. This may occur in cohesive soil during or immediately after construction. Usually, short-term stability of embankments on soft cohesive soil is more critical than long-term stability, because the foundation soil will gain shear strength as the pore pressures dissipate. It may be necessary to check the stability for various pore pressure conditions. Usually, the critical failure surface is tangent to a firm layer underlying the soft soil.

6.3.2.2 CUT SLOPES

The stability of cut slopes made in soft cohesive soils depends on the strength of the soil, the slope angle of the cut, the depth of the excavation, and the depth to a firm stratum (if one exists not too far below the bottom of the excavation). The stability of cut slopes in granular soil is highly influenced by the ground water level and friction angle.

Cut slope failure in soil may result from the following:

- Changes in slope profile, which increases driving forces and/or a decreases resisting forces. Additional embankment on top, steeper side slopes, or undercutting of the toe are examples.
- An increase of pore water pressure, resulting in a decrease in frictional resistance in cohesionless soils, or swell in cohesive soils. An increase in pore pressure could result from slope saturation by precipitation, seepage, or a rise in the ground water elevation.
- Progressive decreases in shear strength due to weathering, erosion, leaching, opening of cracks and fissures, softening, and gradual shear strain (creep).
- Vibrations induced by earthquakes, blasting, or pile driving.
- Earth slopes subjected to periodic submersion (for example, along streams subject to water fluctuations). Also, loss of integrity due to seepage water moving to the face of the cut (piping).

6.3.2.3 ROCK SLOPES:

In addition to the above failures in cut slopes involving rock and/or soil may result from:

- Chemical weathering
- Freezing and thawing of water in the joints
- Seismic shock
- Increase in water pressure within the discontinuities
- Alternate wetting and drying (especially in expansive shales)
- Increase in tensile stress, due to differential erosion

6.3.3 DISCUSSION

While an analysis by hand is very helpful in understanding the mechanics of sliding earth masses such analysis is time consuming. Computer aided procedures are available and they provide a far more detailed analysis in less time.

There are also rules of thumb that can be used to make a preliminary assessment of the Factor of Safety (FOS) to prevent failure. One such rule is: (Taylor's equation)

$$FOS = \frac{6C}{\gamma H}$$

Where: C = cohesion of soft foundation soil
γ = unit weight of embankment soil
H = Height of slope

The FOS computed using the above equation should not be used for final design. This simple equation can be used to preliminarily check both slope and foundation (base) stability. If the factor of safety is less than 2.5, a more sophisticated stability analysis is required. A number of slope stability methods of analysis have been adapted for use with a computer, and without a doubt, there will be others in the future. The concern is whether or not the computer program represents the short-term and long-term conditions that exist in the field. For those analyses, the problem is described by a two-dimensional slice, and the slice is typically thin (such as 1 ft. thick). The program should have the capacity to represent the actual site conditions, by inclusion of all forces acting on each side. Some methods include the side forces on each slide, while other methods ignore these forces.

Factor of Safety (FOS) computations shall be made for various assumed failure surfaces until an apparent minimum factor of safety has been established for each analysis. All models will be approved by INDOT prior to performing the analysis. A computer program should be used for analysis. The printout of input data, output data and plot of failure surfaces should be included with the analysis. In case of surcharge loading a graph of surcharge height and pore pressure should be provided.

6.4 INDIVIDUAL PILE ANALYSIS

Deep foundations are defined as piles, drilled shafts, etc. There are numerous static methods available to estimate the ultimate bearing capacity for piles. Although most of these methods are based on the same basic theories, seldom will any two give the same computed capacity. In fact, owing to the wide range of values and assumptions stated in those methods, major discrepancies in the computed capacity sometimes result. In addition, methods that have not been universally accepted are difficult to review and compare with actual field tests.

It is for the above reasons that the INDOT Office of Geotechnical Services is recommending that all Geotechnical Consultants review the methods, assumptions and values used by the INDOT OGS to compute the nominal bearing capacity for piles. The Geotechnical Consultants should analyze both steel encased concrete piles and steel H-piles for most projects. The following approach for calculating the nominal bearing capacity will be used in checking the nominal bearing capacities computed by INDOT's Geotechnical Consultants.

The pile capacity should be determined using the computer program DRIVEN or equivalent which uses Nordlund's and Tomlinson's methods for cohesionless and cohesive soils respectively. A summary of the theory of these two methods is given below. <u>A factor of safety of 2.5 should be used</u> to calculate the pile capacity with these methods.

The nominal capacity (Q_{ult}) of all driven piles may be expressed in terms of skin resistance (Q_s) and point resistance (Q_p);

$$Q_n = Q_S + Q_P$$

Equation (6.4)

The value of both (Q_s) and (Q_P) is determined in each layer based on either frictional or cohesive behavior of the soil. The strength of frictional soils is based on friction angle. Cohesive soil strength is based on undrained shear strength. The pile capacity of cohesive soil layers should not be computed with both friction angle and cohesion values.

When performing pile analyses please make note that the maximum nominal soil, geotechnical resistance shall be based on the following attached table. The nominal driving resistance may exceed these limits for friction piles if proven by a drivability analyses. It is not necessary to address the structural design in the geotechnical report.

Maximum Nominal Soil Resistance $R_{n\ max}$ (Geotechnical Axial Capacities) for Common Piles

Pile Type	Section Area	Maximum Nominal Soil Resistance $R_{n\ max}$
	Inch. sq	Kips
10x42 HP	12.4	341
10x57 HP	16.8	462
12x53 HP	15.5	426
12x63 HP	18.4	506
12x74 HP	21.8	600
12x84 HP	24.6	677
14x73 HP	21.4	589
14x89 HP	26.1	718
14x102 HP	30.0	825
14x117 HP	34.4	946
14" Pipe pile SEC***	***	420
16" Pipe pile SEC***	***	480

Notes: ***Please note the resistance factor, Φ_{dyn}, for calculating the pile geotechnical capacities by the field methods. (With PDA Φ_{dyn} = 0.70 and with gates formula Φ_{dyn} = 0.55)***

*** The maximum nominal capacity and the maximum factored capacity shall be dependent on drivability and the shell thickness. The minimum shell thickness shall be 0.25 inch for 14" O.D and 0.312" for 16" O.D.

The maximum nominal soil resistance can be taken from the above table. From this value back calculate the maximum factored soil resistance with applicable geotechnical losses.

The maximum nominal driving resistance shall be calculated from the maximum nominal soil resistance with the applicable geotechnical losses included.

Factored design load, Q_F, shall be less than the factored design soil resistance, R_R.

$R_{n\ max}$	Maximum nominal soil resistance, i.e. (geotechnical long term capacity)
$R_{R\ max}$	Maximum factored design soil resistance
$R_{ndr\ max}$	Maximum nominal driving resistance
R_n	Nominal soil resistance equal to or less than the $R_{n\ max}$ (Long term capacity)
R_R	Factored design soil resistance equal to or less than the $R_{R\ max}$
R_{ndr}	Nominal driving resistance equal to or less than the $R_{ndr\ max}$

- The resistance factor, Φ*dyn, for calculating the piles geotechnical capacities by means of field methods, shall be taken for PDA as 0.70,* or in Gates' formula as 0.55.
- For a pipe pile, the maximum nominal capacity and the maximum factored capacity shall be dependent on drivability and shell thickness. The minimum shell thickness shall be 0.25. for a 14-in. O.D. pile, or 0.312 in. for a 16-un. O.D. pile.
- From $R_{n\ max}$ shown in the table, back calculate $R_{n\ max}$ with the applicable geotechnical losses.
- $R_{ndr\ max}$ shall be calculated from $R_{ndr\ max}$ with the applicable geotechnical losses included.
- The factored design load, QF, shall be less than RR.

For piles seated on bedrock with minimal penetration in rock, driven through soils, and with less difficulty of driving, a drivability analyses is not required. The structural resistance will control the design. The nominal soil resistance for H piles driven to hard rock may be increased to 65 percent of the nominal structural resistance, P n, if approved by the Office of Geotechnical Engineering.

<u>6.4.1 SKIN RESISTANCE IN GRANULAR SOILS</u>

Determine Q_s for estimating pile quantities as follows (Nordlund's Method). This can be done with DRIVEN.

This method is based on correlation with actual pile load tests results. The pile shape and material are important factors included in this method.

$$Q_s = \sum_{O}^{D} K_\delta C_F P_d \frac{\sin(\omega + \delta)}{COS\omega} C_d \Delta_d \qquad \text{Equation (6.5)}$$

Which simplifies for non-tapered piles (ω = 0) to the following:

$$Q_s = \sum_{o}^{D} K_\delta C_F P_d \sin\delta C_d \Delta_d \qquad \text{Equation (6.6)}$$

Where: Q_s = Total skin friction capacity
K_δ = Dimensionless factor relating normal stress and Effective overburden pressure
P_d = Effective overburden pressure at the center of depth Increment d
ω = Angle of pile taper measured from the vertical
δ = Friction angle on the surface of sliding
C_d = Pile perimeter
Δ_d = Depth increment below ground surface
C_F = Correction factor for K_δ when $\delta \neq Ø$ (soil friction angle)

To avoid numerical integration, computations may be performed for pile segments of constant diameter ($\omega = 0$) within soil layers of the same effective unit weight and friction angle. Then equation (5.5) becomes:

$$q_s = K_\delta C_F P_d \sin\delta C_d D \qquad \text{Equation (6.7)}$$

Where within the segment selected:

P_d	=	average effective overburden pressure in segment D
C_d	=	average pile perimeter
D	=	segment length
q_s	=	capacity of pile segment D (skin friction)

Equation 4 can be more easily understood if skin friction is related to the shear strength of granular soil, i.e., normal force times tangent of friction angle, N tan Ø. In equation 4 the term $K_\delta C_F P_d$ represents the normal force against the pile, Sin δ represents the coefficient of friction between the pile and soil, and C_d D is the surface area in contact with the soil. In effect equation 4 is a summation of the N Tan Ø sharing resistance against the sides of the pile.

Computational Steps for Non-Tapered Piles

1) Draw the existing effective overburden pressure (P_o) diagram.
2) Choose a trial pile length.
3) Subdivide the pile according to changes in the unit weight or soil friction angle (Ø).
4) Compute the average volume per foot of each segment.
5) Enter Figure 6.4 with that volume and the pile type to determine δ / Ø and compute δ.
6) Enter the appropriate chart(s) in Figures 6.5 thru 6.8 to determine K for Ø.
7) If δ ≠ Ø, enter Figure 6.9 with Ø and δ / Ǿ to determine a correction factor C_F to be applied to K_δ.
8) Determine the average values of effective overburden pressure and pule perimeter for each pile segment.
9) Compute q_s from Equation 6.7 for all pile segments and sum to find the ultimate frictional resistance developed by the pile.

For tapered piles Figures 6.5 thru 6.8 must be entered with both Ǿ and ω to determine K_δ. Also, equation 6.4 should be used to compute the capacity. It is recommended that Nordlund's original paper in the May 1963 ASCE Journal (SMF) be referred to for numerical examples of tapered pile static analysis.

Selection of design friction angle should be done conservatively for piles embedded in coarse granular deposits. Pile load tests indicate that predicted skin friction is often overestimated; particularly in soil deposits containing either uniform sized or rounded particles. A conservative approach is to limit the shearing resistance by neglecting interlock forces. This results in maximum friction angle in predominately gravel deposits of 32° for soft or rounded particles and 36° for hard angular deposits. This method also tends to over predict capacity for piles larger than 24 inches in nominal width. The angle of internal friction for cohesionless soils should be limited to a maximum of 36^{o} in the driven program.

6.4.2 END BEARING CAPACITY IN GRANULAR SOILS

Determine Q_p for estimating pile quantities as follows (Thurman's Method). This can be done with DRIVEN:

$$Q_P = A_P \alpha P_d N'_q \qquad \text{Equation (6.8)}$$

Where:

Q_p = end bearing capacity
A_p = pile end area
α = dimensionless factor dependent on depth-width relationship (see Figure 6.10)
P_d = effective overburden pressure at the pile point
N'_q = bearing capacity factor from Figure 5.10

The Q_p value is limited due to soil arching, which occurs around the pile point as the depth of tip embedment increases. For this reason, Nordlund has suggested limiting the overburden pressure at the pile point, P_d to 3000 psf. More recently, the authors have suggested that further limitations must be placed on the end bearing so as not to compute unrealistic values. Therefore, the Q_p value computed from the equation should be checked against the limiting value, Q_{LIM} obtained from the product of the pile end area and the limiting point resistance (q_L) in Figure 6.11. The end bearing capacity should be taken as the less of Q_p or Q_{LIM}.

The actual steel area should be used to calculate and point resistance in the cohesionless soils.

6.4.3 NOMINAL PILE CAPACITY IN GRANULAR SOILS

The nominal capacity of a pile (Q_N), in granular soils can be determined by summing the total frictional resistance (Q_S) and the maximum and bearing resistance (Q_P) as previously stated in Equation 5.4. However, for foundation design only sum those q_s values which are below the deepest soil layer not considered suitable to permanently support the pile foundation. For scour piles, only sum those q_s values below the anticipated scour depth.

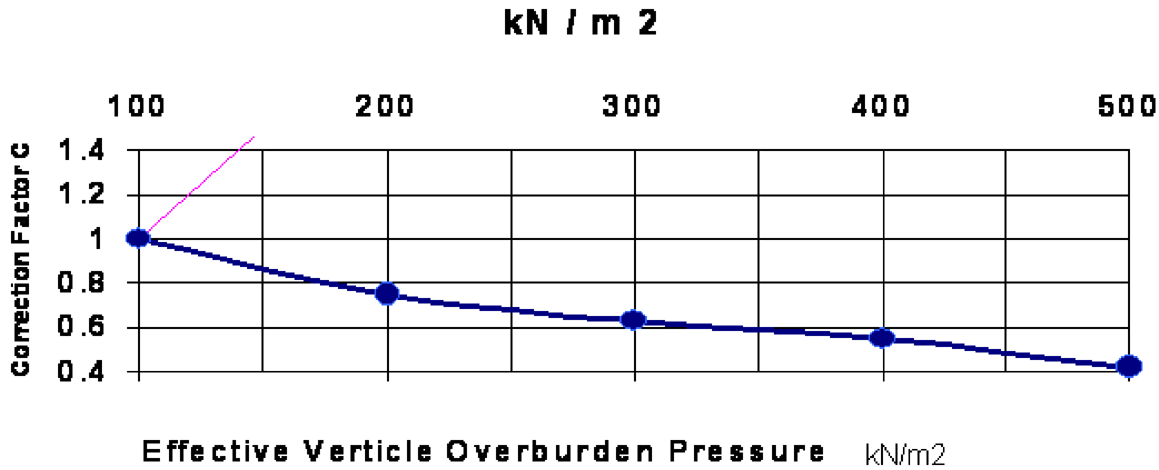
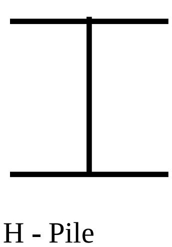


Figure 6.1: Chart For Correction Of N-Values In Sand For Influence Of Overburden Pressure--Reference Value Of Effective Overburden Pressure Of 100 Kn/m2 (1.0 tons/sq ft) (Modified from Peck, et.al., 1979)

Figure 6.2 Suggested End Areas for Driven H and Pipe Piles Where Plug Will Form.

Figure 6.3 Suggested End Areas for Driven H-Pile Where Plug Will Not Form



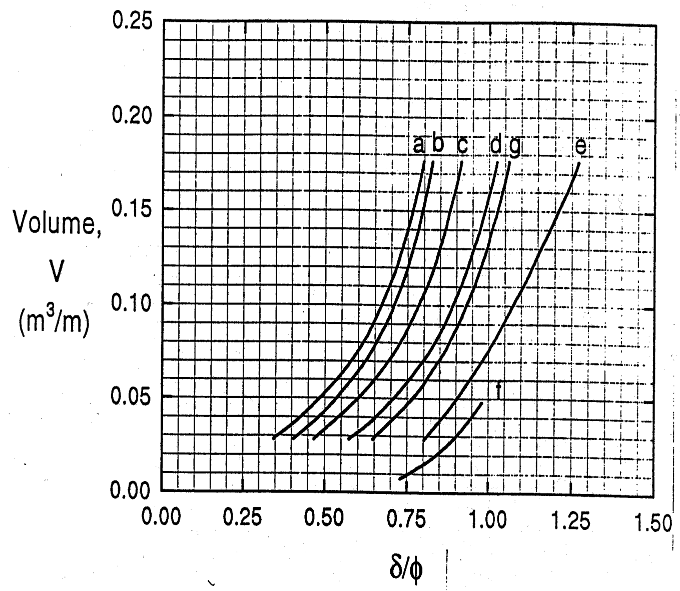


Figure 6.4 Relation of δ/Ø and Pile Displacement . V. for Various Types of Piles

a. Pipe piles and non-tapered portion of monotube piles.
b. Timber piles.
c. Pre-cast concrete piles.
d. Raymond step-taper piles.
e. Raymond Uniform taper piles.
f. H-piles
g. Tapered portion of monotube piles.

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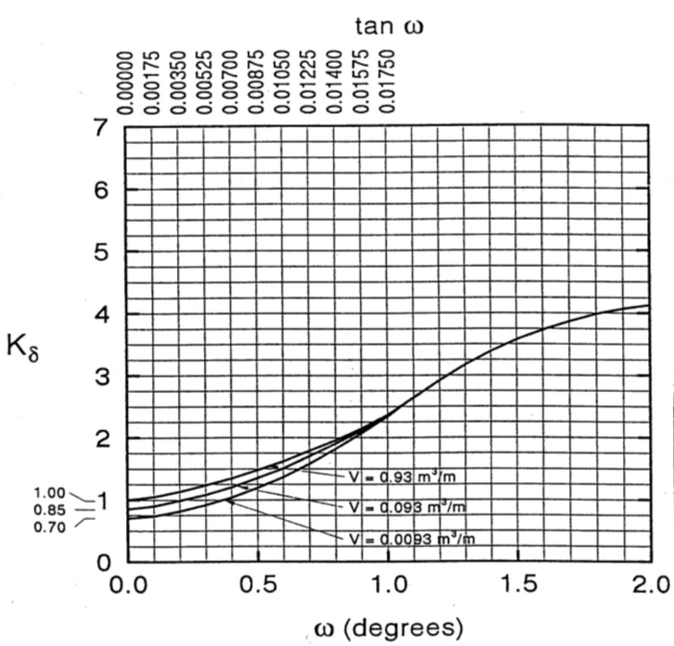


Figure 6.5 Design Curves for Evaluating K_δ for Piles when Ø = 25º (After Nordlund 1979).

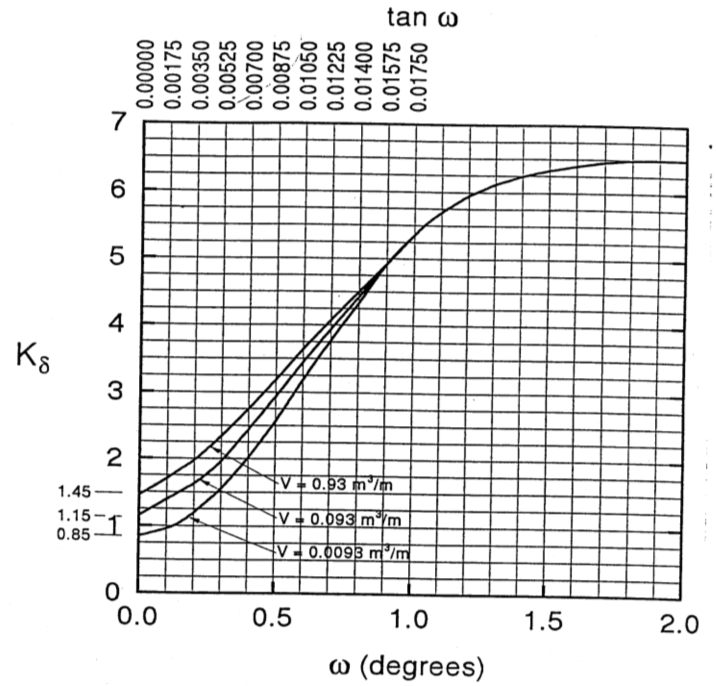


Figure 6.6 Design Curves for Evaluating K_δ for Piles when Ø = 30º (After Nordlund 1979).

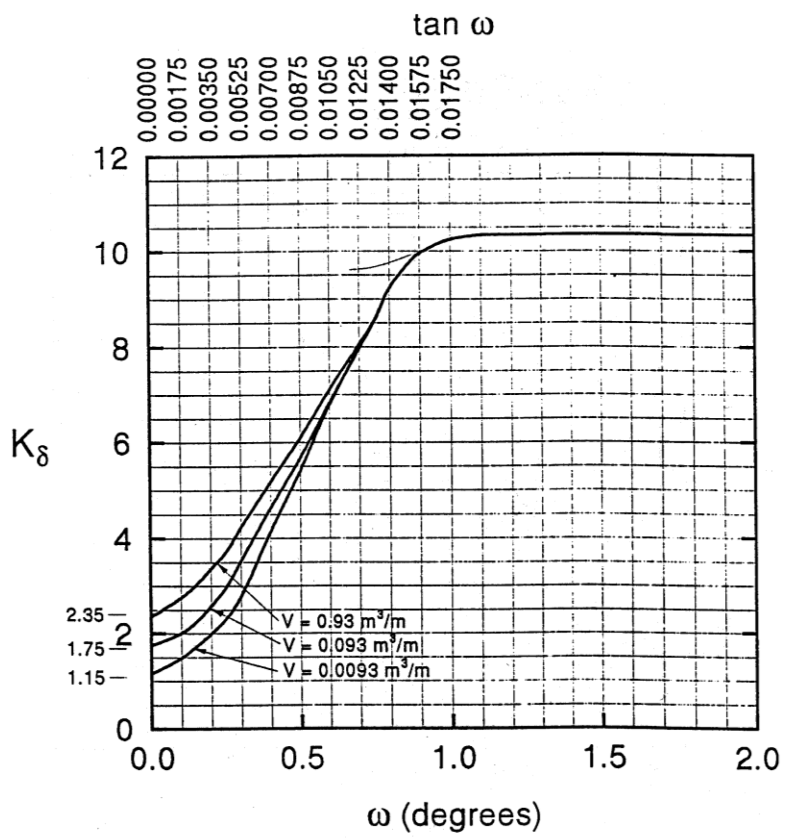


Figure 6.7 Design Curves for Evaluating K_δ for Piles when Ø = 35° (After Nordlund 1979).

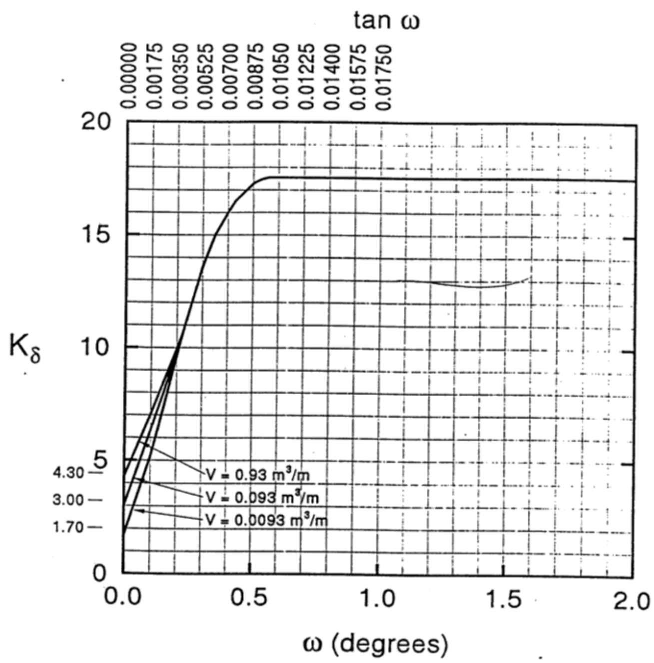


Figure 6.8 Design Curves for Evaluating K_δ for Piles when Ø = 40° (After Nordlund 1979).

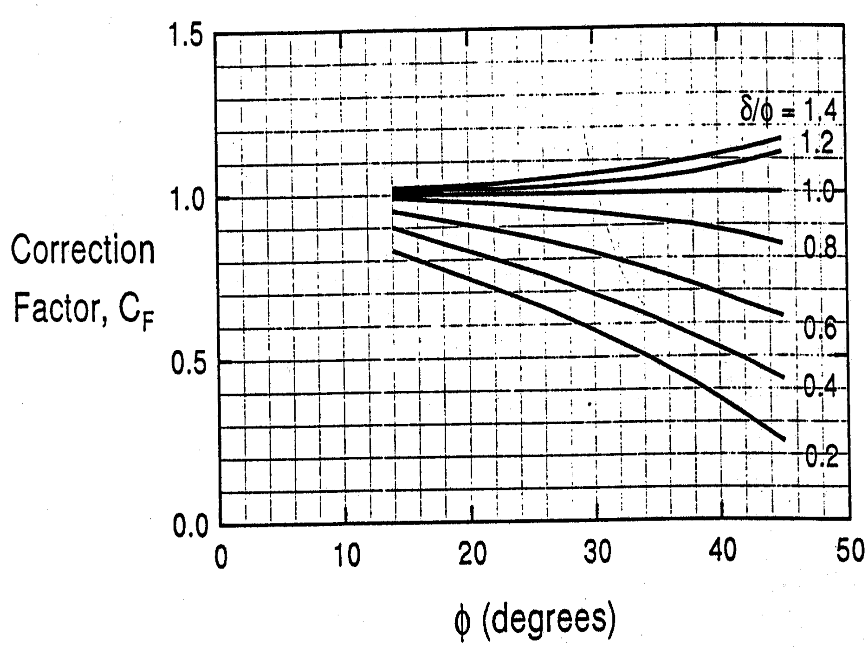


Figure 6.9 Correction Factor for Kδ when ≠ Ø

6.4.4 SKIN FRICTION RESISTANCE IN COHESIVE SOILS

The skin friction resistance for piles which are driven into cohesive soils is frequently larger than eighty (80%) or ninety (90%) percent of the total bearing capacity. Therefore, for such piles, it is extremely important that the skin friction resistance be estimated accurately. Design methods for piles in cohesive soils are in some cases of doubtful reliability. This is particularly true for the load capacity of friction piles in clays of medium to high shear strength (Cu > 100 kN/m^2 (2,000 lb/sq ft).

The frictional resistance is the average friction of adhesion multiplied by the surface area of the pile. For estimation of pile quantities, skin friction may be calculated as:

$$Q_{sf} = f_s PL \qquad \text{Equation (6.9)}$$

where:

f_s = average unit skin friction or adhesion in tsf (KN/m^2)

P = perimeter of the pile (in ft.)

L = embedded length of the pile (in ft.)

The shearing stress between the pile and soil at failure is usually termed the "adhesion" (ca). The average nominal unit skin friction (fs) in homogeneous saturated clay, is expressed by:

$$f_s = c_a = \alpha c_u \qquad \text{Equation (6.10)}$$

In this application, ∝ equals the empirical adhesion coefficient for reduction of average undrained shear strength (c_u) of undisturbed clay within the embedded length of the pile. This method is known as the "Tomlinson Method" or the "∝ Method".

The coefficient ∝ depends on the nature and strength of the clay, pile dimension, method of pile installation and time effects. The values of ∝ vary within wide limits and decrease rapidly with increasing shear strength. The values of ∝ can be obtained from Figure 6.12.

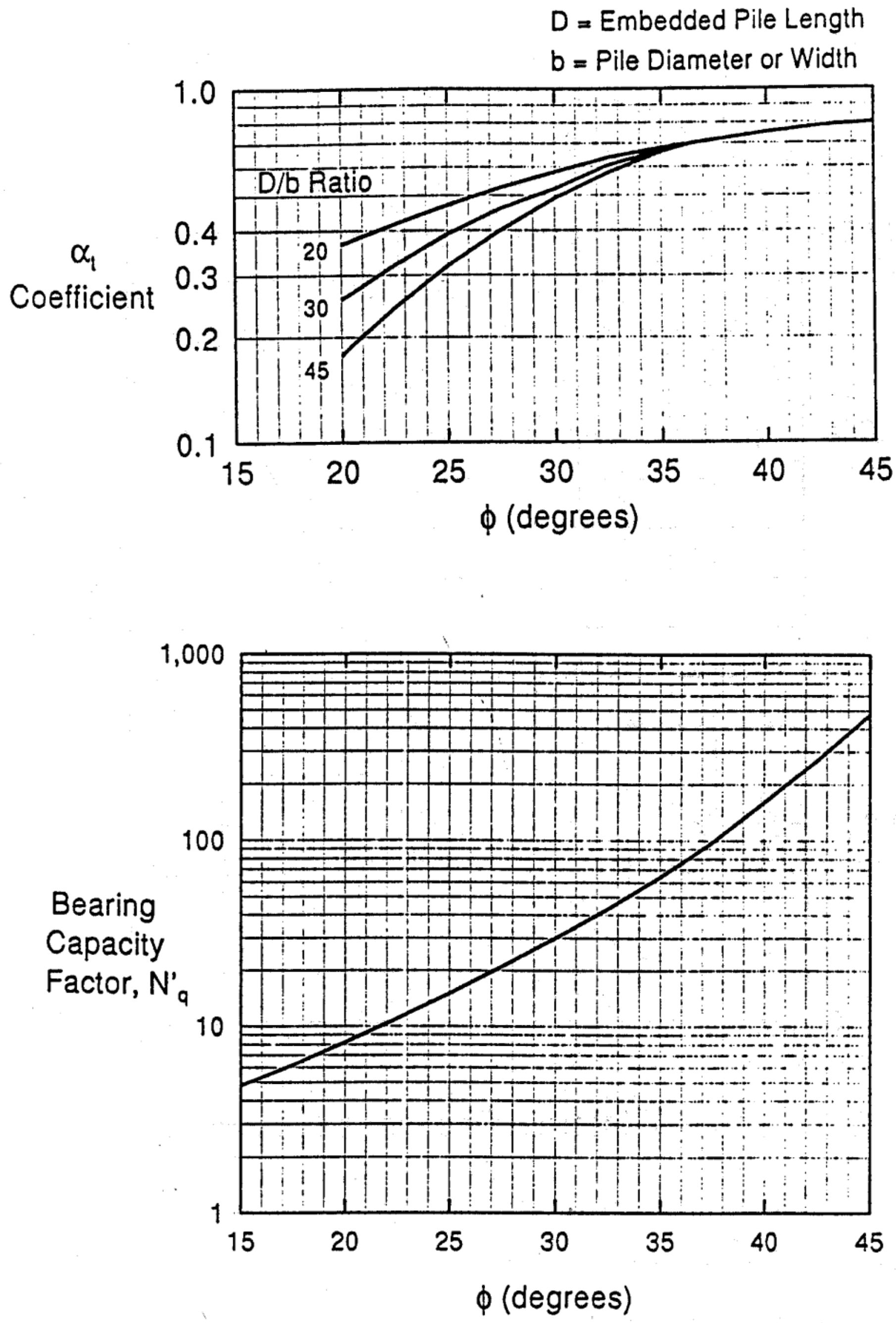


Figure 6.10 Determination of ∝ Coefficient and Variation of Bearing Capacity Factors with ∅

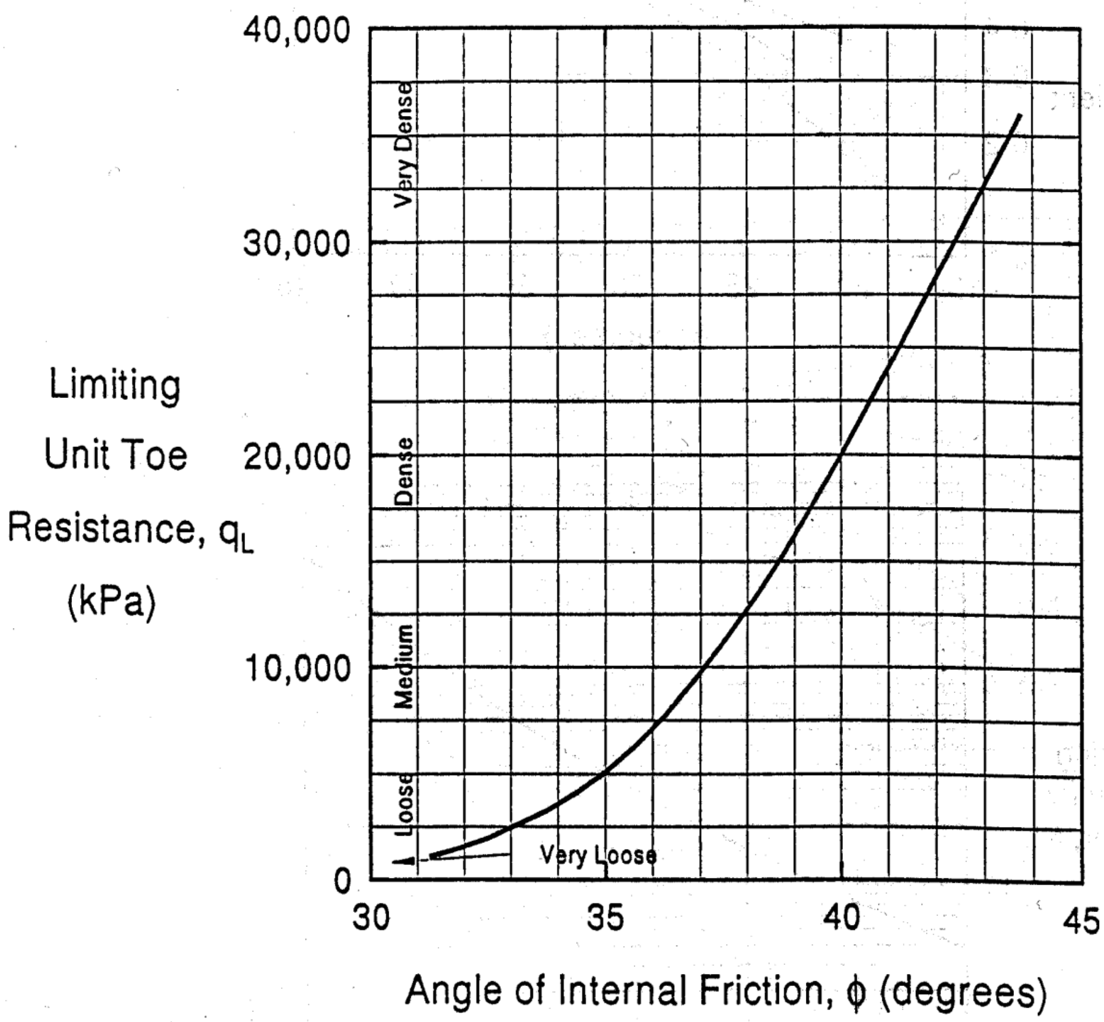


Figure 6.11 Relationship Between Maximum Unit Pile Point Resistance and Friction Angle for Cohesionless Soils (After Meyerhof, 1976)

Shaft resistance is calculated from the sum of the adhesion c_a along the exterior of the two flanges plus the undrained shear strength of the soil, c_u times the surface area of the two remaining sides of the box due to soil to soil shear along these two faces.

Determining Skin Friction Resistance Using The "$\propto$ Method"

STEP 1: Determine adhesion factor $\propto$ from Figure 6.12.

Enter Figure 6.12 with pile length in clay and undrained shear strength of soil (c_u) in psf. Use appropriate curves for situations (a), (b), or (c) shown in the figure.

STEP 2: Compute ultimate unit skin friction resistance (f_s).
$f_s = c_a$ (adhesión) = ($\propto$) x (c_u).

STEP 3: Compute total ultimate skin friction resistance.

$Q_s = (f_s) \times (A_s)$

where: A_s = Pile Surface Area

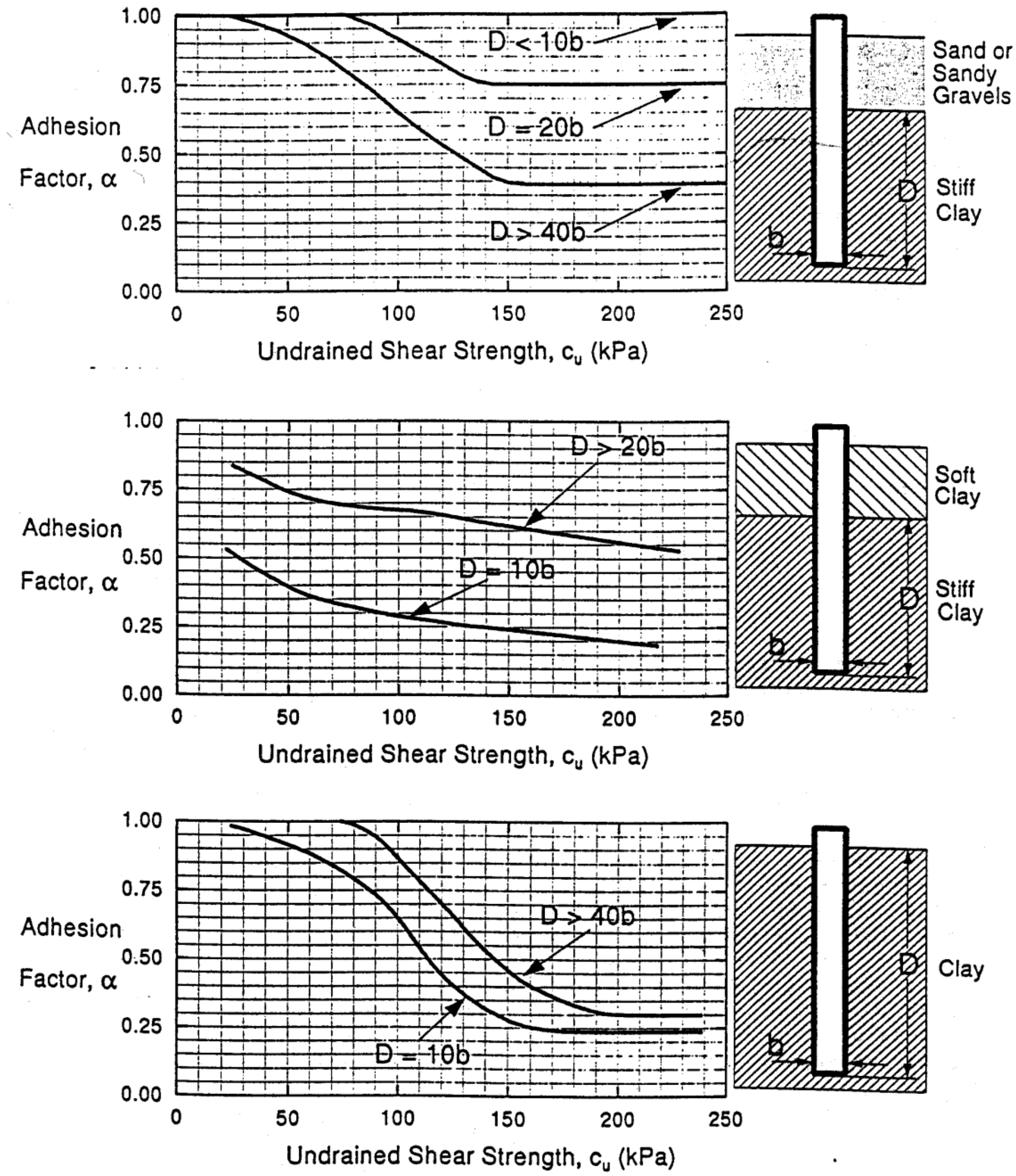


Figure 6.12 Adhesion Factors for Driven Piles In Clay--The Method (After Tomlinson, 1980)

6.4.5 END BEARING CAPACITY IN COHESIVE SOILS

The end bearing component of pile capacity (Q_P) can be determined by the general bearing capacity equation, using factors appropriate for deep foundations:

$$Q_P = Q_P(A_t) = (cNc + PvNQ + 1/2\gamma DN\gamma)A_t \qquad \text{Equation 6.11}$$

where:

Q_P = nominal tip bearing capacity

A_t = area of pile tip

C = undrained shear strength (cohesion) in the vicinity of the tip

γ = effective unit soil weight on the vicinity of the tip

Pv = effective vertical stress (limiting overburden of 10-15 D)

D = pile diameter or width

Nc,Nq,Nγ = deep foundation bearing capacity factors (see Figure 6.13).

NOTE: since D is usually small, the Nγ term is often neglected

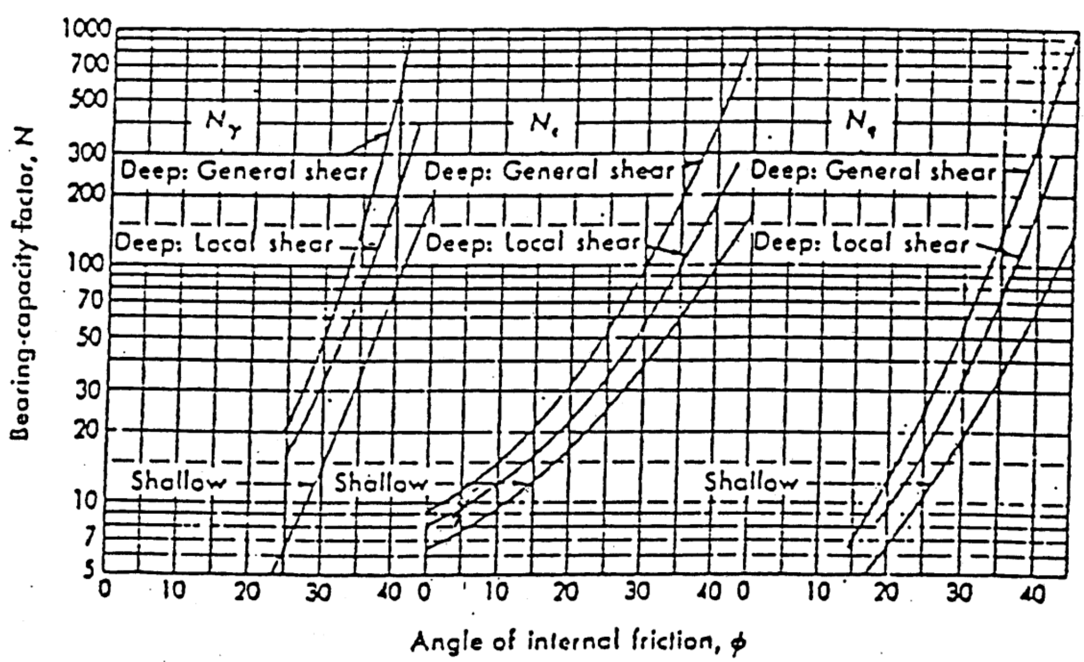


Figure 6.13 Bearing Capacity Factors For Shallow And Deep Square Or Cylindrical Foundations

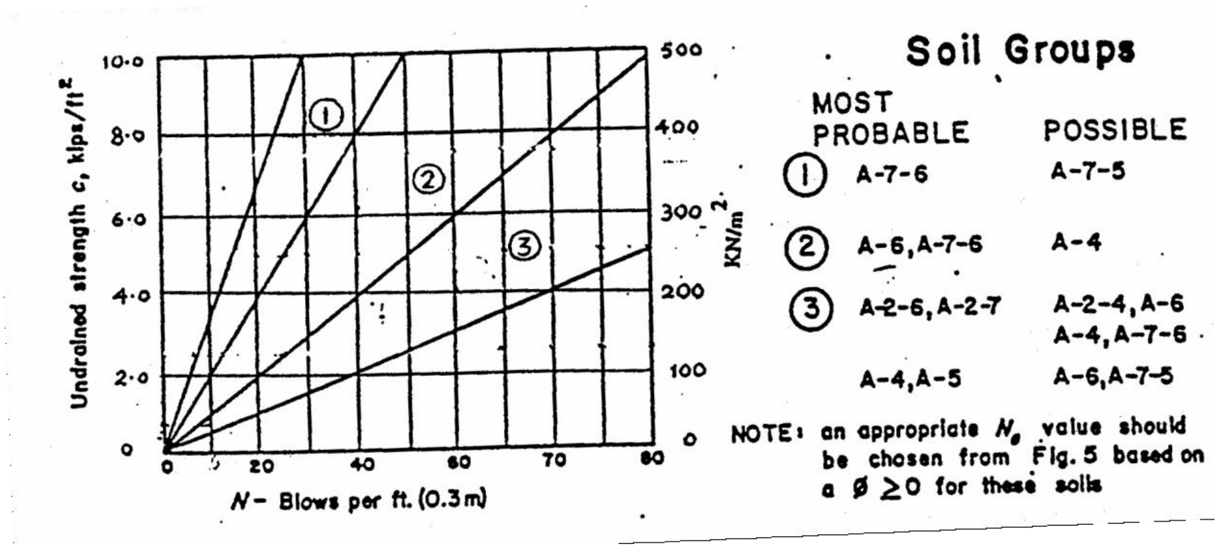


Figure 6.14 Relationship Between Undrained Shear Strength (c) and Penetration Resistance (N)
(Modified After Soweres, 1979)

For clay soils ($\varphi = 0$, and assume Nq = 0), the end bearing becomes:

$$Q_p = cN_c A_t$$ Equation (6.12)

The undrained shear strength (c) of the soil near the sides of the pile and the tip of the pile should be determined in the laboratory. Figure 6.14 correlates the penetration resistance (N) to the undrained shear strength (c) based on textural classification. These are useful correlation for preliminary estimates only.

A soil plug may form at the pile tip and the point bearing capacity may be calculated using the gross cross sectional area (ie. flange width times web depth for H-pile, etc.). This design assumption should be made based upon the subsurface information obtained during the Geotechnical Investigation performed for the project.

6.4.6 NOMINAL PILE CAPACITY IN COHESIVE SOILS

The nominal capacity of a pile (Q_W), in clay can be determined by summing the total frictional resistance (Q_s) and the maximum and bearing resistance (Q_P) as previously stated in Equation 6.4.

6.4.7 PILES IN TILL MATERIAL

Glacial till is composed of unstratified materials that were deposited beneath glacial ice. Over one-half of Indiana is underlain by glacial till. Some layers in the glacial till are referred to as "hardpan" because of the difficulty experienced in driving, drilling, or digging through the material.

The end bearing parameters (N_c & c) for glacial till should be large so that the nominal bearing capacity for driven piles will be obtained in the upper portion of the till. Piles should be driven only a few feet into glacial "hardpan". If the till is predominately non-cohesive, Thurman's end bearing formula (Equation 6.8) should be used.

6.4.8 ADDITIONAL CONSIDERATIONS

Following the analysis of static capacity of single piles, there are many other items requiring consideration (Schroeder, 1970) such as:

- Capacity can change with time.
- Load transfer can change with time from such causes as creep induced by new fill, lowering the groundwater table, remolding of clay, etc.
- Settlement of pile, etc.
- Application to capacity and settlement of pile group
- Negative skin friction, which is a bearing capacity problem induced by settlement. Some causes are:
    - Placement of clay fill over sand where the fill drags the pile down during consolidation and lateral stresses also increase in sand.
    - Placement of fill over compressible clay where fill causes down drag and clay also causes down drag due to consolidation effects.
    - Lowering of the groundwater table in compressible soils.

The method, assumptions, values, etc. presented are based on driven straight steel piles. Drilled or tapered piles and those made of other materials (timber, concrete, etc.) were not considered.

If the pile tip rests in a stratum underlain by a weak soil, the nominal point resistance will be reduced. The nominal point resistance in the bearing stratum will be governed by the resistance to punching of the pile into the underlying weak soil.

6.4.9 PILES ON ROCK

Approximately one-half of the area of Indiana has sedimentary rock near the ground surface (within fifty (50) feet or less). Deep foundations on rock are common where the soil layers are inadequate to support the service load of the structure. The items listed below should be considered for exploration and design for rock foundations.

- Steel Encased Concrete (SEC) piles should not be considered when a deep foundation is to be supported on shale or any other rock. H-piles driven to sound rock should be recommended. Piles on shale should be spaced at a minimum of 6 feet apart.

- Pile tips should not be placed over shallow caves or other large voids. Geologic literature for the area should be reviewed and a detailed field inspection should be performed in areas underlain by limestone.

- Pile tips should not be placed on or stop in coal.

- Rock Quality Designation (RQD) values can provide a qualitative assessment of rock mass as shown in Table 6.2. The RQD is computed by summing the length of all pieces of core equal to or longer than four (4) inches, dividing by the total length of the coring run and multiplying by one-hundred per cent (100 %). Breaks caused by the coring operation should not be used in determining the RQD.

Table 6.2 Engineering Classification For In-Situ Rock Quality Using The Rock Quality Designation (RQD).

RQD %	Rock Mass Quality
90 – 100	Excellent
75 – 90	Good
50 – 75	Fair
25 – 50	Poor
0 – 25	Very Poor

6.4.10 SCOUR DEPTH

The expected scour depth should be considered for every bridge structure over water, unless the scour is protected. The engineer should design the permanent pile capacity to mobilize the required soil resistance below the scour depth. The minimum pile tip elevation, for piers exposed to scour, will be ten (10) feet below the calculated scour depth. For end bents with spill-through slopes the minimum pile tip elevation will be at least equal to the flow line.

The depth of scour (as shown on the plans) is dependent upon the hydrology of the channel, the alteration of the existing channel's cross-section by the proposed bridge structure and the engineering properties of the materials below the stream bed. The Geotechnical Engineer will use the scour depth in the engineering analysis. The scour depth for Q_{100} is generally considered in the engineering analysis.

6.5 PILE GROUP CAPACITY

If Pile Group Capacity analysis is required on a given project, approval must be obtained from the INDOT OGS prior to performing this work. If piles are driven into cohesive/compressible soil or in dense cohesionless material underlain by cohesive/compressible soil, then the load capacity of a pile group may be less than that of the sum of the individual piles. Also, settlement of the pile group is likely to be many times greater than that of an individual pile under the same load. Figure 6.15 shows that only a small zone of soil around and below a single pile is subjected to vertical stress. Figure 6.16 shows that a considerable depth of soil around and below a pile group is stressed and settlement of the whole group may be large depending on the soil profile. The larger zone of heavily stressed soil for a pile group is the result of overlapping stress zones of individual piles in the group. The overlapping effect is illustrated in Figure 6.17. The group efficiency is defined as the ratio of the ultimate load capacity of a group to the sum of the individual ultimate pile load capacities.

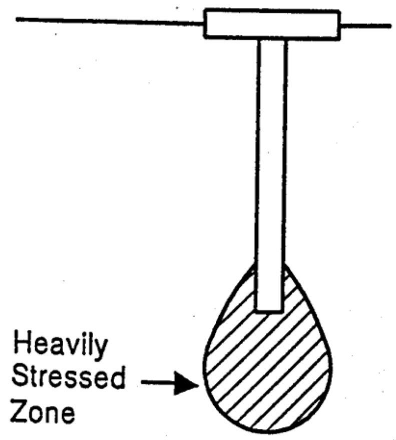


Figure 6.15 Stressed Zone Under End Bearing Single Pile

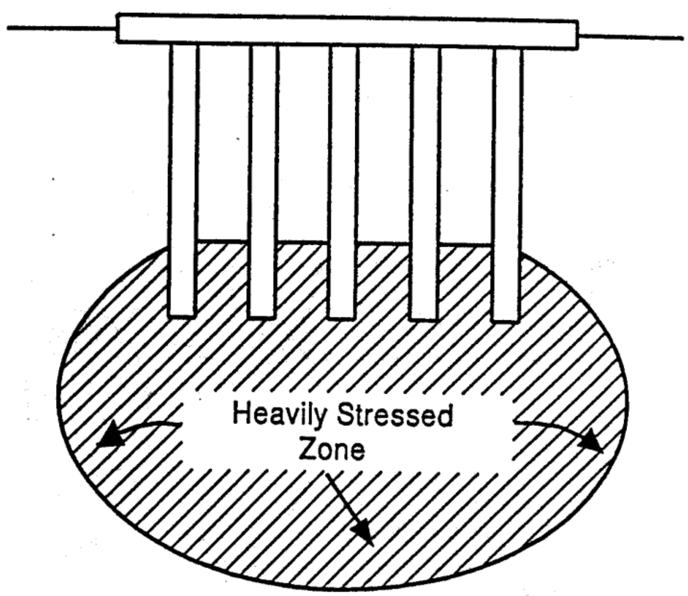


Figure 6.16 Stressed Zone Under End Bearing Pile Group

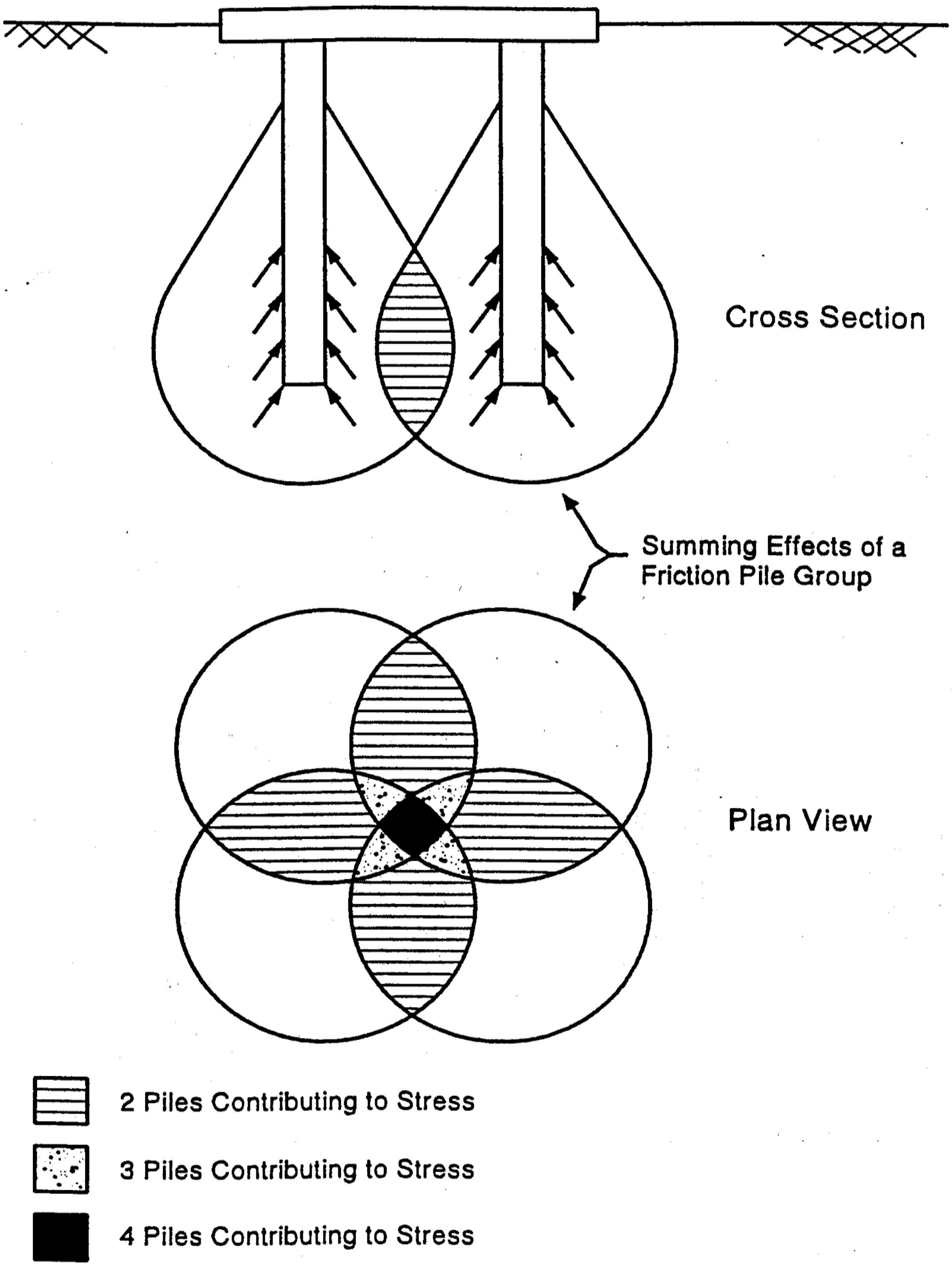


Figure 6.17 Overlapping Stressed Soil Areas For A Pile Group

6.5.1 PILE GROUP CAPACITY IN COHESIONLESS SOILS

In cohesionless soils, the nominal group load capacity of driven piles with a center spacing of less than three pile diameters is greater than the sum of the nominal load of the single piles. The greater group capacity is due to the overlap of individual soil compaction zones near the pile which increases skin resistance. Piles in groups at spacing greater than three times the average pile diameter act as individual piles.

The following are design recommendations for estimating group capacity in cohesionless soil:

- The nominal group load in soil not underlain by a weak deposit should be taken as the sum of the single pile capacities.
- If a group founded in a firm bearing stratum of limited thickness is underlain by a weak deposit, the nominal group load is given by the smaller value of either:
  - The sum of the single pile capacities or,
  - 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 can only occur when the pile spacing is less than two pile diameters, which is rarely the case. The method shown for cohesive soils (in the next section) may be used to investigate the possibility of a block failure.
- Piles in groups should not be installed at spacings less than three times the average pile diameter.

6.5.2 PILE GROUP CAPACITY IN COHESIVE SOILS

In the absence of negative skin friction, the group capacity in cohesive soil is usually governed by the sum of the single pile capacities with some reduction due to overlapping zones of shear deformation in the surrounding soil.

The following are design recommendations for estimating group capacity in cohesive soils:

- For pile groups driven in clays with undrained shear strengths of less than 2,000 psf and for spacings of three times the average pile diameter, the group efficiency can be taken to be equal to seventy percent (70%). If the spacing is greater than six times the average pile diameter, then a group efficiency equal to one-hundred percent (100%) can be used. For additional details, please consult the current NHI and FHWA manuals on pile group capacity.
- For pile groups in clays with undrained shear strength in excess of 2,000 psf, use a group efficiency equal to one-hundred percent (100%).
- Investigate the possibility of a block failure. Recommended method is described in the next section.
- Piles should not be installed at spacings less than three times the average pile diameter in cohesive soils.

6.5.3 NOMINAL RESISTANCE AGAINST BLOCK FAILURE OF PILE GROUP IN COHESIVE SOIL

A pile group in cohesive soil is shown in Figure 6.18. The ultimate resistance of the pile group against a block failure is provided by the following expression:

$$Q_n = (9\,x\,c_{u1}\,x\,B\,x\,L) + (2\,x\,D\,x\,(B+L)\,x\,c_{u2}) \qquad \text{Equation (6.13)}$$

where:

Q_n = Nominal resistance against block failure

C_{u1} = Undrained shear strength of clay below pile tips

C_{u2} = Average undrained shear strength of clay around the group

B = Width of group

L = Length of group

D = Length of piles

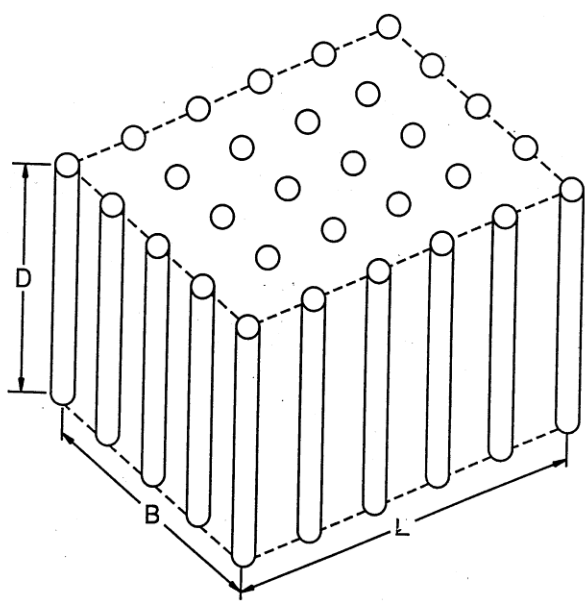


Figure 6.18 Pile Group in Cohesive Soil

6.5.4 SETTLEMENT OF PILE GROUPS

Pile groups supported by cohesionless soils will produce only elastic (immediate) settlements. This means the settlements in cohesionless soils will occur immediately as the pile group is loaded. Pile groups supported by cohesive soils may produce both elastic (immediate) and consolidation (occurs over a time period) settlements. The elastic settlements will generally be the major amount for over-consolidated clays and consolidation settlements will generally be the major amount for 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 because piles are usually installed in groups.

6.5.4.1 SETTLEMENT CAUSED BY ELASTIC COMPRESSION OF PILE MATERIAL DUE TO IMPOSED AXIAL LOAD

The methods discussed in the following sections do not include the settlement caused by elastic compression of pile material due to the imposed axial load. However, this compression can be computed by the following equation:

$$\delta = \frac{\text{P x L}}{\text{A x E}} \qquad \text{Equation (6.14)}$$

where:

δ = Elastic compression of the pile material (usually quite small and is usually neglected in design)

P = Axial load in pile

L = Length of pile

A = Pile cross sectional area

E = Modulus of Elasticity of pile material {E for steel piles = 206843 MPa (30,000,000 psi) and E for concrete piles = 20684 MPa (3,000,000 psi)}.

NOTE: Because the elastic compression of the pile is usually very small, it is often neglected.

6.5.4.2 IMMEDIATE SETTLEMENTS OF PILE GROUPS IN COHESIONLESS SOILS

Meyerhof (1976) recommended that the settlement of a pile group in a homogeneous sand deposit not underlain by a more compressible soil at a greater depth may be conservatively estimated by the following equation:

$$S = \frac{2p\,(B)^{1/2}\,I}{N'} \qquad \text{Equation (6.15a) (English)}$$

Or

$$S = \frac{95p\,(B)^{1/2}\,I}{N'} \qquad \text{Equation (6.15a) (Metric)}$$

For silty sand use the following equation:

$$S = \frac{4p\,(B)^{1/2}\,I}{N'} \qquad \text{Equation (6.15b) (English)}$$

or

$$S = \frac{190p\,(B)^{1/2}\,I}{N'} \qquad \text{Equation (6.15b) (Metric)}$$

where:

S = estimated total settlement in mm (inches)

B = the width of pile group in meter (feet)

p = the foundation pressure in kN/m^2 (tons per square foot) equal to design load to be applied to the pile group divided by the group area

N' = the average corrected SPT resistance (Figure 1) in blows per 0.3 m (foot) within a depth equal to B below the pile tips

I = influence factor for group embedment

= 1 - D / (8 B) > 0.5

D = pile embedment depth, in meter (feet)

6.5.4.3 SETTLEMENT OF PILE GROUPS IN COHESIVE SOILS

A method proposed by Terzaghi and Peck, and confirmed by limited field observations, is recommended for the evaluation of the consolidation settlement of pile groups in cohesive soil. The load carried by the pile group is assumed to be transferred to the soil through a theoretical footing located at 1/3 the pile length up from the pile point (Figure 6.19). The load is assumed to spread within the frustum of a pyramid of side slopes at thirty degrees (30^o) and to cause uniform additional vertical pressure at lower levels, the pressure at any level being equal to the load carried by the group divided by the cross-sectional area of the base of the frustum at that level. This method can be used for vertical or batter pile groups.

The consolidation settlement of cohesive soil is usually computed on the basis of laboratory tests. The relationships of the compression index (Cc) to void ratio e and pressure are shown in Figure 6.20 which is plotted from consolidation test results. For loadings less than the preconsolidation pressure (p_c) settlement is computed using a value of the compression index representing recompression (C_{cr}). For loadings greater than the preconsolidation pressure, settlement is computed using the compression index (C_c).

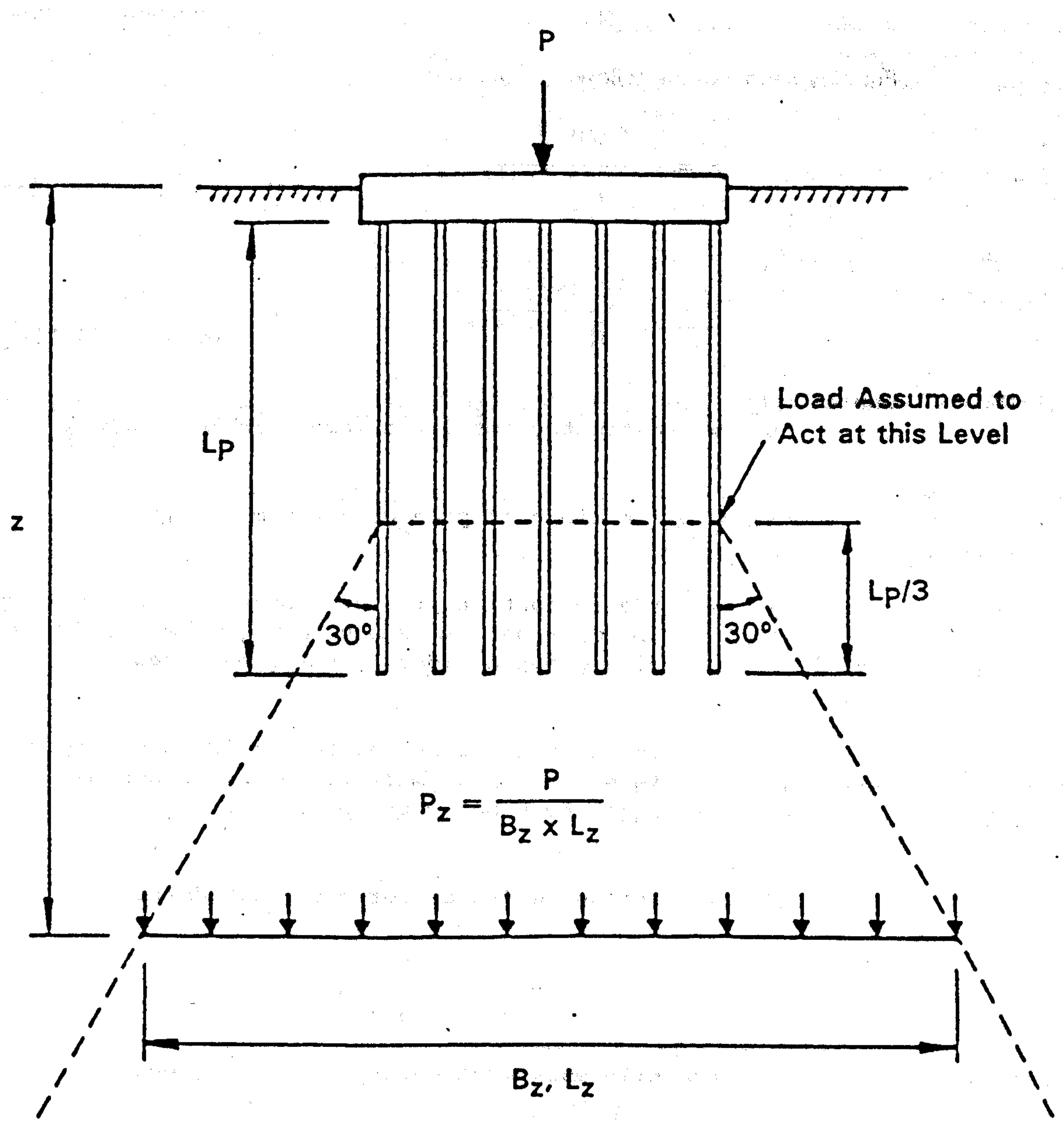


Figure 6.19 Stress Distribution Beneath Pile Group in Clay Using Theoretical Footing Concept
(After Canadian Geotechnical Society, 1978)

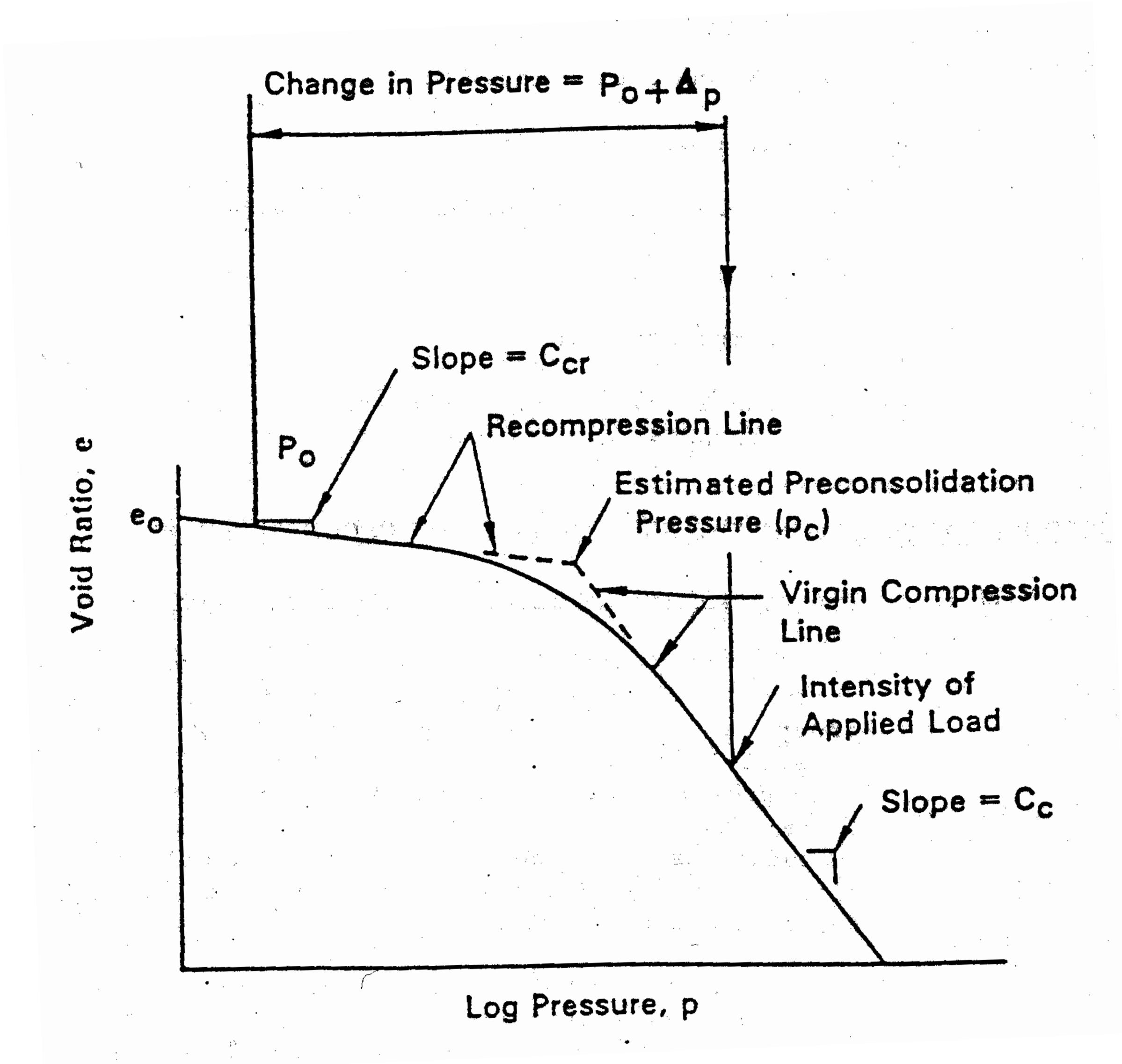


Figure 6.20 The e-log-p Relationship (Modified from Canadian Geotechnical Society, 1978)

The following settlement equation is used for computing consolidation settlement:

$$S = H [(C_{cr} / (1 + e_o)) \log (p_c / p_o) + ((C_c / (1 + e_o)) \log ((P_o + \Delta P) / P_c)] \quad \text{Equation 6.16}$$

where:

S = total settlement

H = original thickness of stratum

C_{cr} = recompression index

e_o = initial void ratio

p_o = average initial effective pressure

p_c = estimated preconsolidation pressure

C_c = compression index

Δp = the average change in pressure in compressible stratum considered

Procedure for Estimating Pile Group Settlement in Cohesive Soil

STEP 1: Determine Load Imposed on the Soil by Pile Group

- Use the method shown in Figure 6.19 to determine the depth at which the additional imposed load by the pile group is less than ten percent (10%) of existing effective overburden pressure at that depth. This will provide the total thickness of cohesive soil layer to be used in performing settlement computations. Use design load to be applied to the pile group. Do not use ultimate pile group capacity for settlement computations.

- Divide the cohesive soil layer determined in 1) above into several thinner layers 1.5 to 3.0 m (five to ten feet) thick. The layer thickness H is the thickness of each layer.

- Determine the existing effective overburden pressure (p_o) at midpoint of each layer.

- Determine the imposed pressure (p) at midpoint of each layer by using the method shown in Figure 6.19.

STEP 2: Determine Consolidation Test Parameters

- Plot results of consolidation test (Figure 6.20)

- Determine p_c, e_o, C_{cr} and C_c from the plotted data.

STEP 3: Compute Settlements

- By using the settlement equation, compute settlement of each layer.

- Summation of settlements of all layers will provide the total estimated settlement for the pile group.

6.6 NEGATIVE SKIN FRICTION

When a soil deposit, through which piles are installed, undergoes consolidation, the resulting downward movement of the soil around piles induces "downdrag" forces on the piles. These "downdrag" forces are also called negative skin friction. Negative skin friction is the reverse of the usual positive skin friction developed along the pile surface. This force increases the pile axial load and can be especially significant on long piles driven through compressible soils, and must be considered in pile design. Batter piles should be avoided in negative skin friction situations because of the additional bending forces imposed on the piles, which can result in the pile breaking.

Settlement computations should be performed if necessary to determine the amount of settlement the soil surrounding the piles is expected to undergo after the piles are installed. The amount of relative settlement between soils and pile that is necessary to fully mobilize negative skin friction is approximately 0.5 inches. At that movement the maximum value of negative skin friction is equal to the soil adhesion or friction resistance. The negative skin friction can not exceed these values because slip of the soil along the pile occurs at this value. It is particularly important in the design of friction piles to determine the depth below which the pile will be unaffected by negative skin friction. Only below that depth can positive skin friction forces provide support to resist vertical loads. Figure 6.21 shows two situations where negative skin friction may occur. Situation (B) is the most common.

Since negative skin friction is similar to positive skin friction (except that the direction of force is opposite), previously discussed methods can be used for computing pile skin friction.

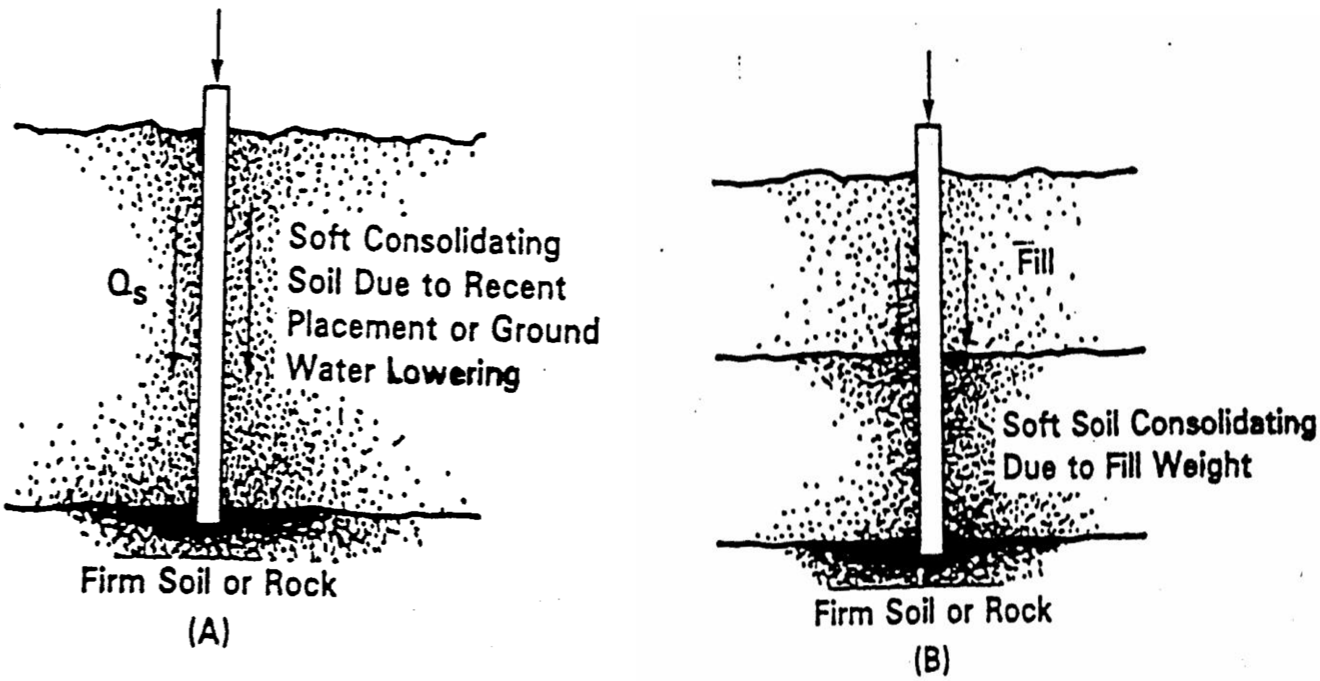


Figure 6.21: Negative Skin Friction Situations

6.7 LATERAL SQUEEZE OF FOUNDATION SOIL

Bridge abutments supported on piles driven through soft cohesive, or compressible, soils may tilt forward or backward depending on the geometry of the backfill and the abutment (Figure 6.22). If the horizontal movement is large, it may cause damage to structures. The unbalanced fill loads shown in Figure 6.22 displace the soil laterally. The lateral displacement may bend the piles, causing the abutment to tilt toward or away from the fill.

The following rules of thumb are recommended for determining whether lateral squeeze or tilting will occur, and estimating the magnitude of horizontal movement involved:

6.7.1 DETERMINING LATERAL SQUEEZE

Lateral squeeze or abutment tilting can occur if:

(γ_{fill} x h_{fill}) > (3 x un drained shear strength of soft soil).

6.7.2 MAGNITUDE OF HORIZONTAL MOVEMENT

If abutment tilting can occur, the magnitude of the horizontal movement can be estimated by the following formula:

Horizontal Abutment Movement = 0.25 x Vertical Fill Settlement

6.7.3 SOLUTIONS TO PREVENT TILTING

The following solutions are possible means of eliminating tilting:

- Allow the fill settlement to occur before abutment piling is installed (best solution)
- Provide expansion shoes large enough to accommodate movement
- Use steel H-piles to provide high tensile strength in flexure
- Excavate the compressible soils and replace with engineered fill

6.8 PILE LATERAL LOADING

Horizontal loads and moments on a vertical pile are resisted by the stiffness of the pile and mobilization of resistance in the surrounding soil as the pile deflects. Following is a description of the parameters used in the determination of lateral load capacity of piles.

6.8.1 SOIL PARAMETERS

- Soil type and physical properties such as shear strength, friction angle, density, and moisture content

- Coefficient of horizontal subgrade reaction (kg/m^3) or (pci). This coefficient is defined as the ratio between a horizontal pressure per unit area of vertical surface (kN/m^2) or (psi) and the corresponding horizontal displacement (m) or (in). For a given deformation, the greater the coefficient, the greater is the lateral load capacity.

6.8.2 PILE PARAMETERS

- Physical properties such as shape, material, and dimensions
- Pile head conditions such as free head or fixed head
- Method of placement such as jetting or driving
- Group action

6.8.3 LOAD PARAMETERS

- Type of loading such as static (continuous) or dynamic (cyclic)
- Eccentricity

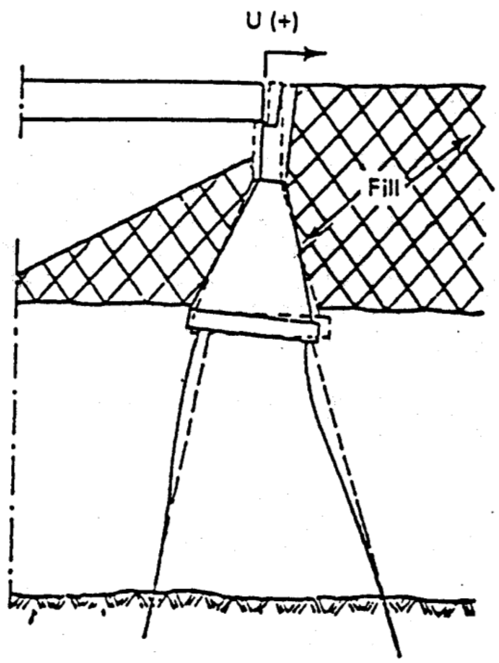


Figure 5.22: Abutment Tilting Due To Lateral Squeeze

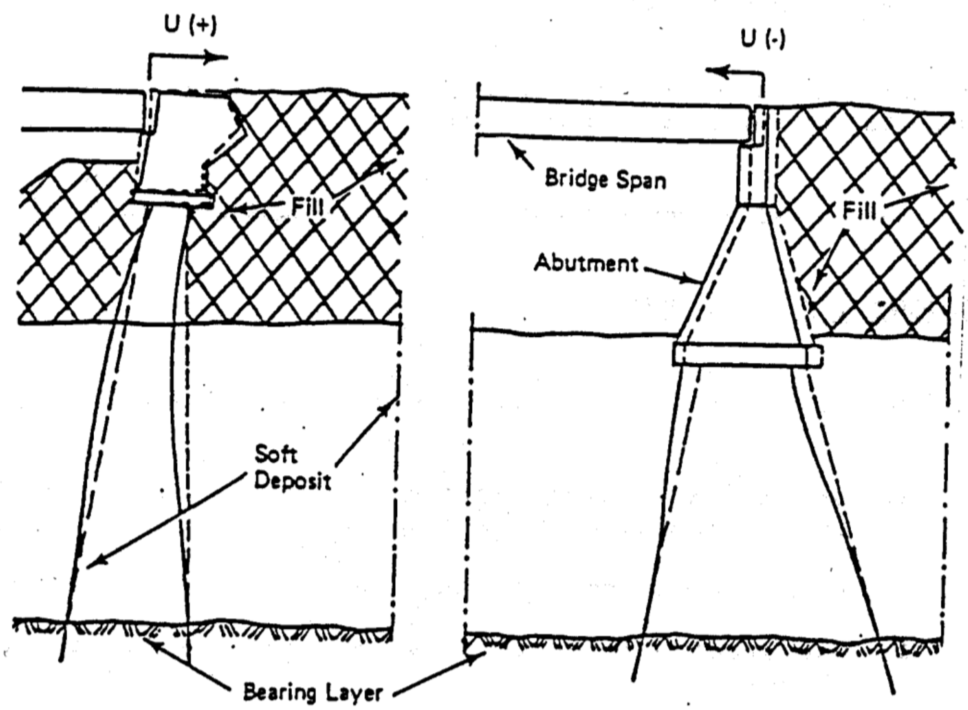


Figure 6.22
(con't.) Abutment Tilting Due To Lateral Squeeze

6.8.4 DESIGN METHODS FOR LATERALLY LOADED PILES

Three basic design approaches are used in practice. They are lateral load tests, arbitrary values, and analytical methods.

6.8.4.1 LATERAL LOAD TESTS

Full scale lateral load tests can be conducted at a construction site during the design stage. The data obtained is used to complete the design for the particular site. These tests are time-consuming, costly and can only be justified on large projects of a critical nature.

Table 6.3. Prescription Values For Allowable Lateral Loads On Vertical Piles (After New York, State Department of Transportation, 1977).

SOURCE	PILE TYPE	DEFLECTION in (mm)	SERVICE LATERAL LOADS lbs	(kg)	
NYSDOT	TIMBER	---	10,000	(4500)	
	CONCRETE	---	15,000	(6800)	
	STEEL	---	20.000	(9000)	
NY CITY 1968 BLDG CODE	ALL	3/8 (10)	2,000	(900)	
TENG	ALL	1/4 (6.5)	SOFT	CLAYS:	
			1,000	(450)	
FEAGIN	TIMBER	1/4 (6.5)	9,000	(4100)	
	TIMBER	1/2 (12.5)	14,000	(6300)	
	CONCRETE	1/4 (6.5)	12,000	(5400)	
	CONCRETE	1/2 (12.5)	17,000	(7700)	
McNULTY	in (mm)		MEDIUM SAND	FINE SAND	MEDIUM CLAY
	12 (300) TIMBER*(FREE)	1/4 (6.5)	1,500 (680)	1,500 (680)	1,500 (680)
	12 (300) TIMBER (FIXED)	1/4 (6.5)	5,000 (2,250)	4,500 (2,000)	4,000 (1,800)
	16 (400) TIMBER (FREE)	1/4 (6.5)	7,000 (3,200)	5,500 (2,500)	5,000 (2,250)
	16 (400) TIMBER (FIXED)	1/4 (6.5)	7,000 (3,200)	5,500 (2,500)	5,000 (2,250)

*SAFETY FACTOR OF 3 INCLUDED

6.8.4.2 ARBITRARY (PRESCRIPTION) VALUES

Arbitrary values of lateral load capacity are empirical. They do not consider all the site parameters and may lead to over-design or under-design. These values should be used only when little or no information exists regarding the specific site. The recommended values by several sources differ widely. The Canadian Foundation Engineering Manual states the following: "For cases of vertical piles subjected to small and transient horizontal loads it is common practice to assume that such piles can sustain horizontal loads up to 10% of the allowable vertical load (service limit, state load) without special analysis or design features" (Becker & Moore, 2006).

6.8.4.3 ANALYTICAL METHODS

The analytical methods are based on theory and empirical data and permit the inclusion of various site parameters. Two available approaches are (1) Brom's method and (2) Wang and Reese's methods. Both approaches consider the pile to be analogous to a beam on an elastic foundation. Brom's method provides a relatively easy, hand calculation procedure to determine lateral loads and pile deflections at the ground surface. Brom's method ignores the axial load on the pile. Wang and Reese's more sophisticated methods include analysis by computer (COM-624 Program) and a non-dimensional method which does not require computer use. Wang and Reese's computer method permits the inclusion of more parameters and provides moment, shear, soil modulus, and soil resistance for the entire length of pile including moments and shears in the above ground sections.

It is recommended that for the design of major pile foundation projects, Wang and Reese's more sophisticated method be used. These methods are described in a FHWA manual on lateral load design (FHWA-IP-84-11). For small scale projects the use of Brom's method is recommended.

A step by step procedure showing the application of Brom's method, developed by the New York State Department of Transportation (1977), is provided below:

STEP 1: General Soil Type:

Determine the general soil type (i.e., cohesive or cohesionless) within the critical depth below the ground surface, approximately four or five pile diameters.

STEP 2: Coefficient of Horizontal Subgrade Reaction:

Determine the coefficient of horizontal subgrade reaction K_h within the critical depth from a cohesive soil:

Cohesive Soils: $$K_h = \frac{n_1 n_2\, 80\, q_u}{D}$$

where:

q_u = unconfined compressive strength in kN/m^2 (psf)
D = width of pile in meter (feet)
η_1 and η_2 = empirical coefficients taken from Table 5.4.

TABLE 6.4. Values of Coefficients n_1 and n_2 For Cohesive Soils.

UNCONFINED COMPRESSIVE STRENGTH (q_u) in kN/m^2 (psf)	n_1
< 50 (1000) 50 (1000) to 200 (4000) > 200 (4000)	0.32 0.36 0.40
PILE MATERIAL	n_2
STEEL CONCRETE WOOD	1.00 1.15 1.30

TABLE 6.5. Values of K_h For Cohesionless Soils

SOIL	K_h in kg/m^3 (lbs/in^3)	
DENSITY	ABOVE GROUND WATER	BELOW GROUND WATER
LOOSE	200 x 10^3 (7)	110 x 10^3 (4)
MEDIUM	830 x 10^3 (30)	550 x 10^3 (20)
DENSE	1800 x 10^3 (65)	1100 x 10^3 (40)

STEP 3: Loading and Soil Conditions:

Adjust K_h for loading and soil conditions:

a) Cyclic loading (for earthquake loading) in cohesionless soil:

1) K_h = 1/2 K_h from Step 2 for medium to dense soil.
2) K_h = 1/4 K_h from Step 2 for loose soil.

b) Static loads resulting from soil creep (cohesive soils):

1) Soft and very soft normally consolidated clays: K_h = (1/3 to 1/6) K_h from Step 2.
2) Stiff to very stiff clays. K_h = (1/4 to 1/2) K_h from Step 2.

STEP 4: Pile Parameter:

Determine the pile parameter:

a) Modulus of elasticity E (kN/m^2) or (psi).
b) Moment of inertia I (m^4) or (in^4).
c) Section modulus S about an axis perpendicular to the load plane (m^3) or (in^3).
d) Yield stress of pile material f_y (kN/m^2) or (psi) for steel or ultimate compression strength f_c (kN/m^2) or (psi) for concrete.
e) Embedded pile length L (m) or (in).

f) Diameter or width D (m) or (in).

g) Eccentricity of applied load e for free-headed pile -- i.e., vertical distance between ground surface and lateral load, (m) or (in).

h) Dimensionless shape factor C_s (steel piles only):
1) Use 1.3 for piles with circular cross-section
2) Use 1.1 for H-section piles when the applied lateral load is in the direction of the pile's maximum resisting moment (normal to pile flanges).
3) Use 1.5 for H-section piles when the applied lateral load is in the direction of pile's minimum resisting moment (parallel to pile flanges).

i) M_{yield}, the resisting moment of the pile = $C_s f_y S$ (M-kg) or (in lb) (for steel piles).
$M_{yield} = f'_c s$ (m-Kg) or (in lb) for concrete piles.

STEP 5: Factor β or n:

Determine factor β or n:

a) $\beta = \sqrt[4]{K_h D / 4EI}$ for cohesive soil, or

b) $n = \sqrt[5]{K_h / EI}$ for cohesionless soil.

STEP 6: The Dimensionless Length Factor:

Determine the dimensionless length factor:

a) β L for cohesive soil, or

b) η L for cohesionless soil.

STEP 7: Determine if the Pile is Long or Short:

a) Cohesive soil

1) $\beta L > 2.25$ (long pile)

2) $\beta L < 2.25$ (short pile)

NOTE: It is suggested that for βL values between 2.0 and 2.5, both long and short pile criteria should be considered in Step 9. Use the smaller value.

b) Cohesionless soil

1) $\eta L > 4.0$ (long pile)

2) $\eta L < 2.0$ (short pile)
3) $2.0 < \eta L < 4.0$ (intermediate pile)

STEP 8: Other Soil Parameters:

Determine other soil parameters:

1) Rankine passive pressure coefficient for cohesionless soil, $K_p = \tan^2 (45 + \phi/2)$ where ϕ = angle of internal friction.
2) Average effective soil unit weight over embedded length of pile γ (kg/m^3) or (pcf).
3) Cohesion, C_u = one-half unconfined compressive strength, ($q_u/2$) (kN/m^2) or (psi).

STEP 9: <u>Nominal (Failure) Load for a Single Pile:</u>

Determine the nominal (failure) load P_u for a single pile:

1) Short Free or Fixed-Headed Pile in Cohesive Soil:

Using L/D (and e/D for the free-headed case), enter Figure 6.24 select the corresponding value of $P_u / C_u D^2$, and solve for P_u (kg) or (lb.).

2) Long Free or Fixed-Headed Pile in Cohesive Soil:

Using $M_{yield} / C_u D^3$ (and e/D for the free-headed case), enter Figure 6.25, select the corresponding value of $P_u/C_u D^2$, and solve for P_u (kg) or (lb.).

3) Short Free or Fixed-Headed Pile (Cohesionless Soil):

Using L/D (and e/L for the free-headed case), enter Figure 6.26, select the corresponding value of $Pu/KpD_3\gamma$ and solve for Pu (kg) or (lb.).

4) Long Free or Fixed-Headed Pile (Cohesionless Soil):

Using $M_{yield} / D^4\gamma Kp$, (and e/D for the free-headed case), enter Figure 6.27, select the corresponding value of $P_u/K_pD^3\gamma$ and solve for P_u (kg) or (lb.).

5) Intermediate Free/Fixed-Headed Pile (Cohesionless Soil):

Calculate P_u for both a short pile (step 9-3) and a long pile (step 9-4) and use the smaller value.

STEP 10: <u>Maximum Service Limit State Load</u>

Calculate the maximum allowable working load for a single pile P_m from the nominal load P_u determined in Step 9.1 (this is shown in Figure 6.27)

$$P_m = \frac{P_u}{2.5} \text{ (kg) or (lb.)}$$

STEP 11: Service Load for a Single Pile for a given Deflection

Calculate the service load for a single pile P_a corresponding to a given design deflection (y), at the ground surface or the deflection, corresponding to a given design load. If P_a and y are not given, substitute the value of P_m (kg) or (lb) (from Step 10) for P_a in the following cases and solve for Y_m (m) or (in.):

1) Free or Fixed-Headed Pile in Cohesionless Soil:

Using βL (and e/L for the free-headed case), enter Figure 6.30 select the corresponding value of $\gamma K_h DL/P_a$, and solve for P_a (kg) or (lb). From P_a you can get y (m) or (in.).

2) Free or Fixed-Headed Pile in Cohesive Soil:

Using ηL (and e/L for the free-headed case), enter Figure 6.23, select the corresponding value of $\gamma\ (E\ I)^{3/5}\ K_h^{2/5}/P_a\ L$, solve for P_a (kg) or (lb). From P_a you can get y (m) or (in.).

STEP 12: If $P_a > P_m$, use P_m and calculate Y_m (Step 11).

If $P_a < P_m$, use P_a and Y.

If P_a and Y are not given, use P_m and Y_m.

STEP 13: Reduce the service load selected in Step 12 to account for:

1) Group effects as determined by pile spacing Z in the direction of load (see Figure 6.30).

2) Method of installation: For jetted piles use 0.75 of the value from Step 13-1). For driven piles use no additional reduction.

STEP 14: The total lateral load capacity of the pile group equals the adjusted service load per pile from Step 13-2) times the number of piles. The deflection of the pile group is the value selected in Step 12. It should be noted that no provision has been made to include the lateral resistance offered by the soil surrounding an embedded pile cap.

Z	REDUCTION FACTOR
8D	1.0
6D	0.8
4D	0.5
3D	0.4

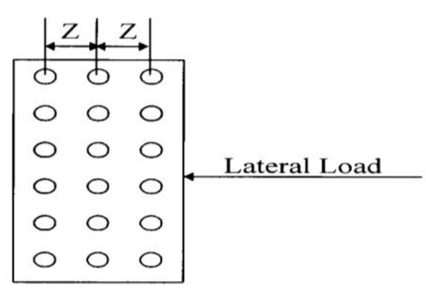


Figure 6.23 Group Effects As Determined By Pile Spacing Z In The Direction Of Load:

Special Note

Inspection of Figures 6.25 and 6.26 for cohesionless soils indicates that the nominal load P_u is directly proportional to the effective soil unit weight. As a result, the ultimate load for short piles in submerged cohesionless soils will be about 50 percent of the value for the same soil in a dry state. For long piles, the reduction in P_u is somewhat less than 50 percent due to the partially offsetting effect that the reduction in γ has on the dimensionless yield factor. In addition to these considerations, it should be noted that the coefficient of horizontal subgrade reaction K_h is less for the submerged case (Table 6.5) and thus the deflection will be greater than for the dry state.

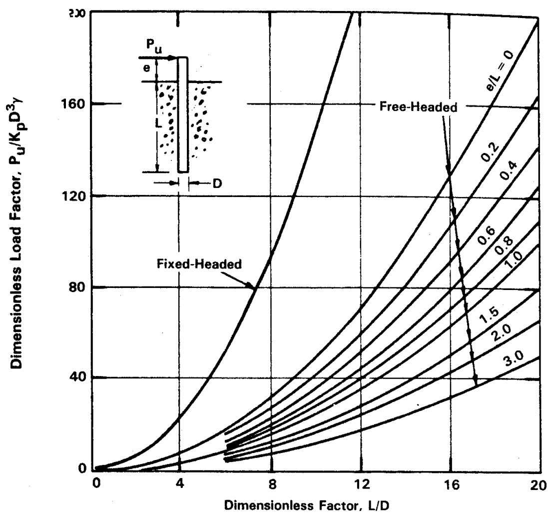


Figure 6.24 Nominal Lateral Load Capacity of Short Piles in Cohesive Soils

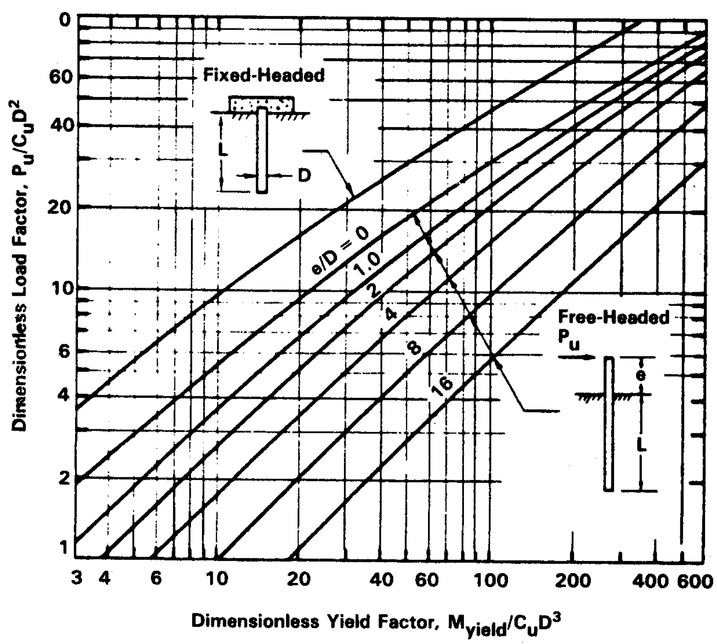


Figure 6.25 Nominal Lateral Load Capacity of Long Piles in Cohesive Soils

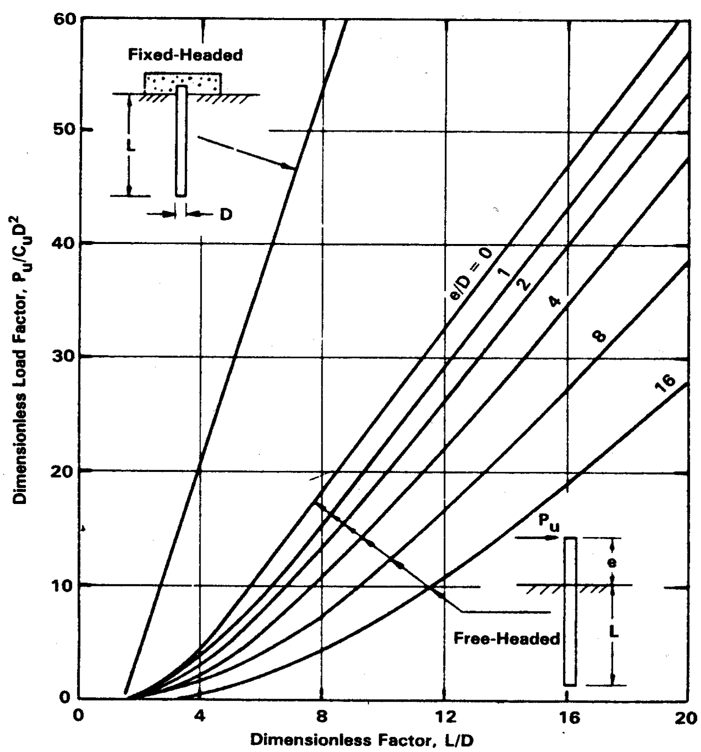
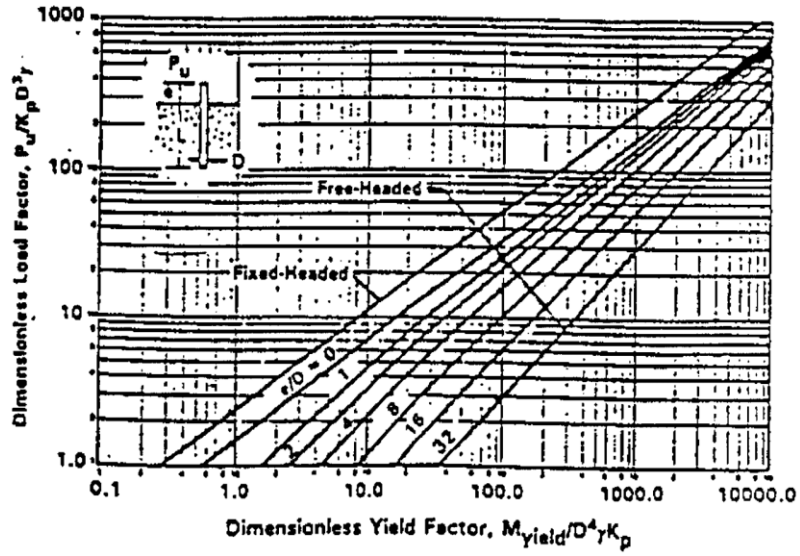
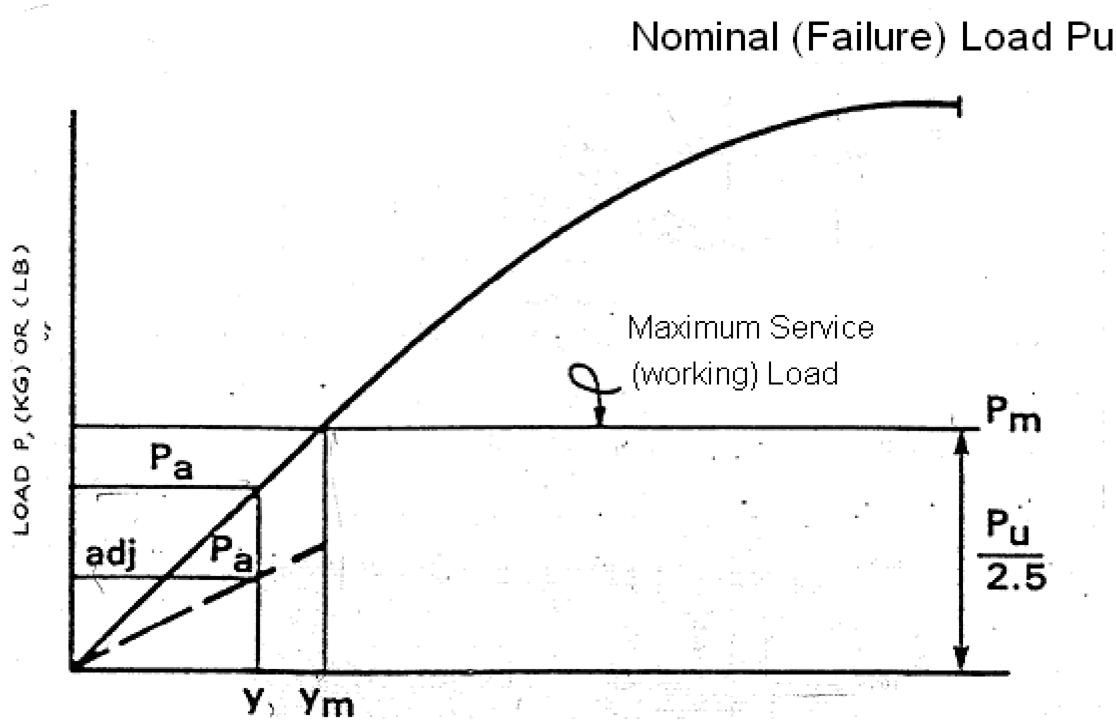


Figure 6.26 Nominal Lateral Load Capacity of Short Piles in Cohesionless Soils


1000
100
10
1.0
0.1
1.0
10.0
100.0
1000.0
10000.0
Dimensionless Load Factor, $P_u/K_pD^3\gamma$
Dimensionless Yield Factor, $M_{yield}/D^4\gamma K_p$
Free-Headed
Fixed-Headed
e/D = 0
1
2
4
8
16
32
P_u
e
L
D

.

Figure 6.27 Nominal Lateral Load Capacity of Long Piles In Cohesionless Soils



Deflection $_y$ (M) or (N)

Figure 6.28 Relationship Between Load and Deflection

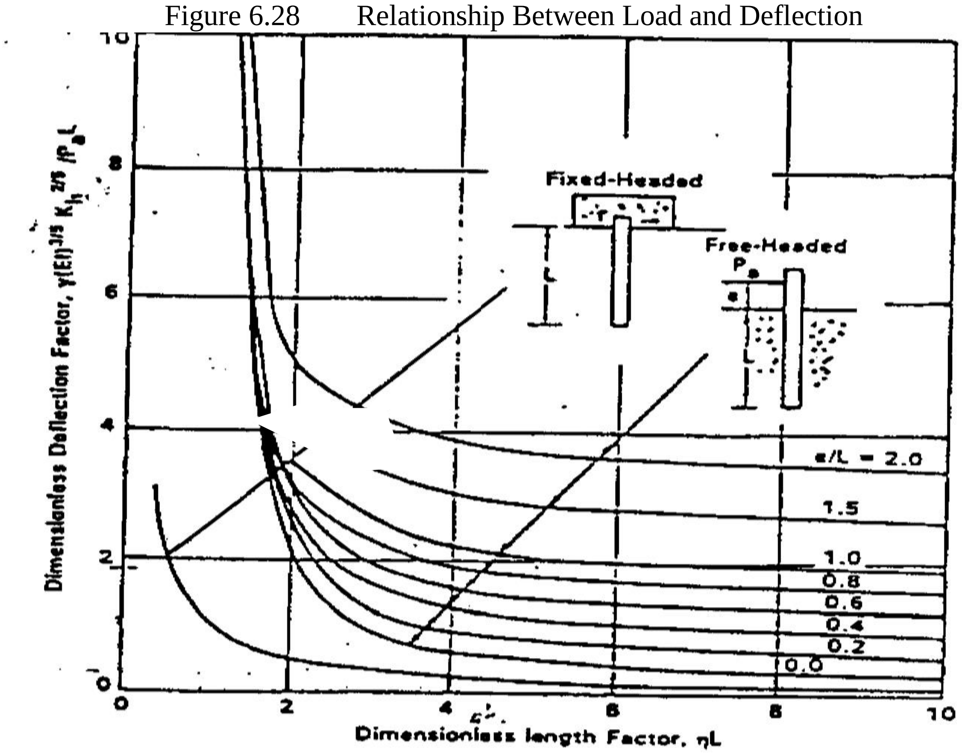


Figure 6.29 Lateral Deflections, At Ground Surface Of Piles in Cohesive Soils

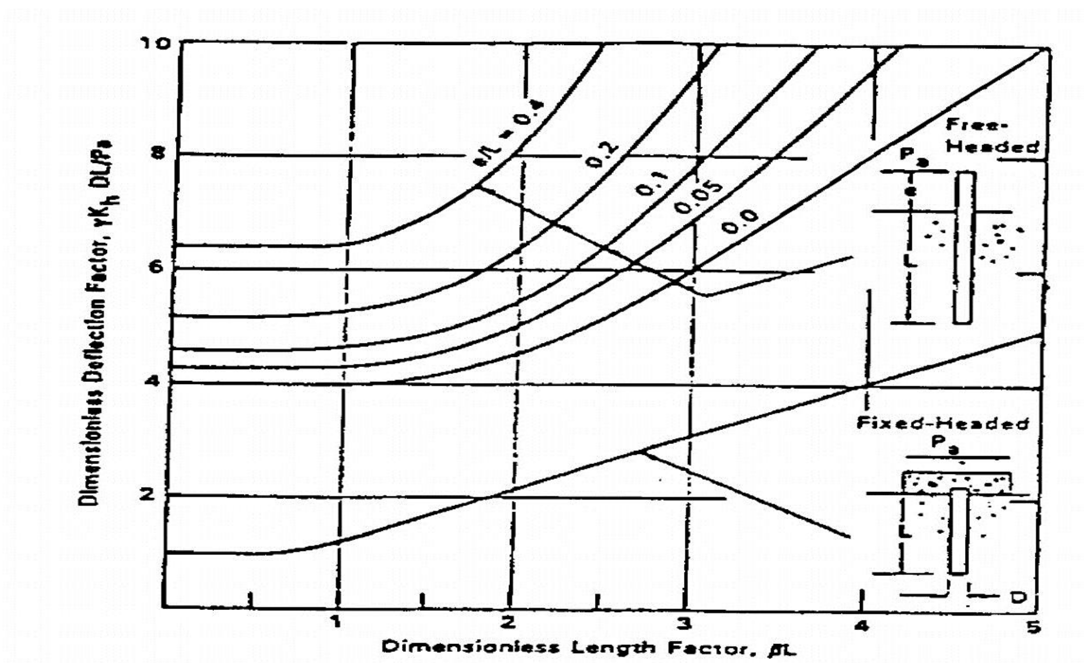


Figure 6.30: Lateral Deflections, at Ground Surface of Piles in Cohesionless Soils

6.9 SEISMIC CONSIDERATIONS

Indiana has a significant seismic history which is concentrated in the southwest part of the State, distance from the earthquake epicenter, site conditions, probable magnitude of the earthquake, etc. should be considered by the geotechnical engineer to determine the seismic effects on proposed foundations in accordance with current memorandums available on the INDOT Website. During subsurface investigation of soil strata consists of significant amount of loose sand (N**<**11), the site should be checked for liquefaction.

6.9.1 Liquefaction Susceptibility Assessment Procedure

In evaluating liquefaction susceptibility/potential, engineering judgment should be used with respect to the age of the deposit, the thickness of the deposit, and whether or not the suspect layer is confined between two low permeability layers. In some cases, some or all of these factors could be used to rationally eliminate a granular layer from liquefaction consideration.

Determine the Site Class and Seismic Zone from the AASHTO Interim Rev. 2008, Section 3.10.1. Site Class should be determined using the AASHTO Method verified by shear wave velocity data and the design ground motion parameters should be derived using Seismic Design Parameters Version 2.10 software (AASHTO/USGS).

If the Seismic Zone is 3 or 4 then a liquefaction assessment shall be conducted. A liquefaction assessment shall also be considered where very loose to loose (e.g., $(N_1)_{60}$ <10 bpf or q_{c1n}<75 ksf) saturated sand exists in Seismic Zone 2 and the Acceleration Coefficient (A_s) is 0.15 or higher.

The procedure for evaluating liquefaction shall be based on AASHTO Interim Rev. 2009 Section 10.5.4.2.:

6.9.1.1 DETERMINE SUSCEPTIBILITY:

In general, only non-plastic soils such as sands or silts will liquefy. However, there are some low plasticity soils that will liquefy too.

1. If the granular soil is present within 75 ft of the ground surface, such as sand, non-plastic silt, or loose gravel, and groundwater is within 50 ft of the ground surface, then continue, otherwise stop.
2. If the soil is cohesive, determine initial susceptibility using the following criteria. If either criterion shows the soil is susceptible to liquefaction then continue, otherwise stop.
    - Boulanger & Idriss (2006) suggest that soils with a PI≥7 **are not susceptible** to liquefaction.
    - Bray & Sancio (2006) suggest that a soil with a PI<12 and a water content to LL ratio (Wc/LL) > 0.85 **will be susceptible** to liquefaction.

6.9.1.2 DETERMINE LIQUEFACTION POTENTIAL:

1. Calculate the liquefaction potential for each sample interval in each granular layer using the Simplified Method from *Semi-Empirical Procedures for Evaluating Liquefaction Potential During Earthquakes* by I.M. Idriss and R.W. Boulanger, January 2004. Where available, CPT data shall be used, otherwise use SPT data in evaluating liquefaction potential. The earthquake moment magnitude (M_W) shall equal 6.5 and the peak ground surface acceleration shall equal A_s [based on Seismic Design Parameters (AASHTO/USGS)]. Where soils are determined to be susceptible to liquefaction, a liquefaction potential analysis shall be performed at each bridge bent/pier.

2. If the Factor of Safety against liquefaction is less than 1.2 (per INDOT, 2/16/2010), then the effects of liquefaction shall be assessed. For Design Build the contractor is responsible for mitigation methods. For Design Bid Build, INDOT's consultant is responsible for the mitigation methods.
3. In reporting liquefaction potential (for Design Build), provide the depth for which mitigation shall be performed.

During liquefaction, pore-water pressure build-up occurs, resulting in a temporary loss of strength and then settlement as excess pore-water pressure dissipates. Potential effects include: slope failure, flow failure or lateral spreading, and downdrag on deep foundations. The design of the mitigation method is the responsibility of the Design/Build Firm and is subject to approval from the Office of Geotechnical Engineering at INDOT.

6.9.2 Geotechnical Seismic Uplift Design Criteria:

For each multi-span bridge structures in Seismic Zones, we understand that lateral loads at the bridge foundations are such that large uplift loads are being generated at interior piers during an extreme event (i.e., seismic load case). The pile skin friction resistance (R_s) should be considered for resistance to uplift.

Per 10.7.3.8.6(a-4), $R_s = q_s * A_s$, where:
q_s = nominal unit side resistance along the length of the pile (psf) which will be provided by the geotechnical consultant for each soil layer; and
A_s = surface area of pile side (sq ft)

A_s is a function of the pile size. In most cases, this is taken as the box perimeter of the pile used in design multiplied by the unit length of the pile. For cases where rock sockets or drilled shafts are considered, A_s will be controlled by the diameter of the rock socket/shaft. For sockets in rock, we recommend that ISS Section 701.09a (2) be used to determine the minimum diameter of a pre-cored hole (pile dia. + 4 in.) and that the skin friction in the overburden soils be neglected. The cored hole diameter could be increased to accommodate for the required uplift resistance.

In the extreme load case, a resistance factor (ø) of 0.8 shall be considered for uplift resistance of piles and shafts. The resistance factor shall be provided in the geotechnical recommendations. For evaluating uplift, the geotechnical engineer shall provide the nominal (unfactored) unit side resistance, q_s, per foot of the pile length.

The structural designer shall include the design unfactored and factored uplift loads and a minimum tip elevation (indicating whether compression or uplift controls) on the Foundation Review form and on the contract plans. The designer should also consider geotechnical losses due to scour and liquefaction if applicable. Soils in liquefiable zones shall not be used for uplift resistance.

6.9.3 Seismic Slope Stability of Embankments

The following procedure as taken from NCHRP Report 611 shall be followed to check for Seismic slope stability:

Step 1: Complete an assessment of Static Slope Stability. The resistance factors or factors of safety shall be as required:

	Min. Factor of Safety	Max. Resistance Factors
Roadway Embankments	1.3	0.75
Approach Embankments at Structures	1.5	0.65

Step 2: Determine Slope Aspect and Site Specific Seismic Coefficients, A_S, SD_S, and SD_1 as per AASHTO

Step 3: Check if liquefaction potential exist at the approach embankments as described in Geotechnical Design Memo #2. If Yes – Mitigation is required. After mitigation is addressed check the criterion in Step 4.

For other roadway embankments if Step 1 is satisfied, and Step 2 determinations are done, proceed to Step 4.

Step 4: Check the no-analyses cut off criteria below:

Slope Angle	A_S	Action
3H:1V	<0.3	No analysis required
2.5H:1V	<0.25	No analysis required
2H:1V	<0.2	No analysis required

If the above criteria are satisfied then no further analyses are required for seismic slope stability.
If the above criteria are not satisfied proceed to Step 5.

Step 5: If the proposed slope fails the above criteria Seismic Slope Stability Analysis is required. Undrained Total Stress/Strength parameters shall be used in the analysis.

The peak ground acceleration used in the analysis shall be defined as:

$A_g = A_S * 0.5 * \alpha$

Where $\alpha = 1 + 0.01H[(0.5\beta)-1]$

H = fill height in feet

$\beta = (F_V S_1)/A_S$

$\alpha = 1$, where slope height is <30 ft

Step 6: Check the Resistance factor or the factor of safety achieved from seismic slope stability analyses. If the resistance factor is less than 0.9 or the factor of safety is greater than 1.1, the slope meets seismic design requirements. If the requirements are not met mitigation must be performed.

6.10 RETAINING STRUCTURES

Retaining structures are an important part of the roadway network. It is often required to retain earthen material at both cut and fill areas, particularly when right-of-way is restricted. Examples of retaining structures are: conventional retaining walls, MSE walls, bin walls, solider-pile walls, modular block walls, soil nailed walls, and drilled-in-pier retaining structures, etc. All retaining walls should be designed according to AASHTO LRFD (2007 [revised 2015] and 2009 interim ed.), FHWA and NHI-05-094 revised copy number is 15-047 guidelines with latest additions.

6.10.1 CONVENTIONAL RETAINING STRUCTURES:

Conventional retaining structures include cantilever walls, bridge abutments, etc. Based on the loading conditions and subsurface soils encountered, the type of foundations (shallow or deep) are decided.

Lateral pressure is the governing factor for the geotechnical design of the retaining structures. Following are the steps to analyze the stability of retaining structures with shallow foundations.

- Sum of all vertical components of loads, ΣV.
- Sum of all horizontal components of loads ΣH.
- Sum of all moments of all vertical and horizontal forces at the toe.

$\sum M_v$ and $\sum M_h$

$$x = \frac{\sum M_v - \sum M_h}{\sum V}$$

For other types of retaining walls use AASHTO LRFD guidelines.

Eccentricity of resultant load at foundation: Use AASHTO LRFD guidelines.
Factored Bearing Capacity: Use AASHTO LRFD guidelines.
Sliding – Use AASHTO LRFD guidelines.
Overturning – Use AASHTO LRFD guidelines.

In case settlement is anticipated within the foundation influence zone due to the presence of soft soil, settlement analysis should be performed. Based on the laboratory test results a plot of total estimated settlement vs time, assuming most likely drainage conditions, should be presented for a specific section (cross sections should be provided).

6.10.2 SOLDIER PILE RETAINING STRUCTURES:

Analysis of these types of structures takes into account

Lateral loads on structures
Depth of embedment for stability
Strain limits of the structural elements and
Soil and/rock pressure against structures.

6.10.3 SOLDIER PILE RETAINING STRUCTURES WITH TIE-BACK SYSTEM:

In addition to the factors described in part (b) above this analysis also includes the capacity of tie backs; the penetration required for stability, the spacing of tie backs and other design parameters. The distance and inclination of tie backs from the top of drilled pier and the amount of maximum movement is determined for each pier.

6.11 SEEPAGE ANALYSIS AND DRAINAGE FILTER REQUIREMENTS

Seepage analysis is conducted at specific sections to estimate the quantity of seepage through and/or underneath the embankment, etc. Stability against piping and any other related analysis are to be analyzed as a part of the seepage analysis. However, prior approval of INDOT must be obtained before performing the analysis.

The Engineer shall furnish computations for estimated seepage, calculated factor of safety against piping and all necessary curves and sketches.

For drainage filter requirements, the following criteria are followed:

- To avoid head loss in the filter: (D_{15} filter ÷ D_{15} protected layer) > 4,, and the permeability of the filter must be adequate for the drainage system.

- To avoid movement of particles from the protected layer:

  (D_{15} filter ÷ D_{85} protected layer) < 5
  (D_{50} filter ÷ D_{50} protected layer) < 25, and
  (D_{15} filter ÷ D_{15} protected layer) < 20:

  - For a very uniform protected layer:

    ($C_u < 1.5$): (D_{15} filter ÷D_{85} protected layer) may be increased to 6.

  - For a broadly graded base material ($C_u > 4$):

    (D_{15} filter ÷ D_{15} protected layer) may be increased to 40.

  NOTE: $C_u = (D_{60} \div D_{10})$ = coefficient of uniformity.

- To avoid movement of the filter into the drain pipe perforation or joints:

  - (D_{85} filter ÷ slot width) > (1.2 to 1.4)
  - (D_{85} filter ÷ hole diameter) > (1.0 to 1.2)

- To avoid segregation, the filter should contain no particle size larger than 3".

- To avoid internal movement of fines, the filter should have no more than 5% passing 0.075 mm (No. 200) sieve.

When the above criteria cannot be satisfied without using a multifilter media, the use of a suitable geotextile can be included with a granular material. In this application, the geotextile may be used to line the trench to protect against the movement of fines into the collector.

6.12 GEOSYNTHETIC REINFORCEMENT

The development of geosynthetics offers a range of new products for providing: 1) tensile characteristics to soils, 2) separation of different particle size materials; 3) filtration to allow movement of water without movement of soil fines; 4) a retaining system; and 5) serving more than a single purpose by employing the products in combination, if necessary. In most cases, geo synthetics (geocell confining system or geomembrane) are used to provide these benefits. However, metal reinforcement has been extensively used in MSE walls.

The use of geosynthetics may expedite construction, enhance stability, and realize economic advantages that do not occur with soil-aggregate systems.

6.12.1 SUBGRADE REINFORCEMENT:

The supporting capacity of subgrade varies widely due to different kind of subgrade soils generally encountered including cohesive and non-cohesive nature. It is very important not to over stress the subgrade for the stability of the pavement. Reinforcement is a very effective option for enhancing the bearing strength of a subgrade. Geosynthetic reinforcement does the following:

- Improves tensile strength of subgrade.
- Spreads the loads in wider area.
- Generally reduce the thickness of the granular material (stone) layer above the subgrade.
- Separates fines and aggregate at interface or prevents intrusion of aggregate into soft subgrade.
- Reduces rutting of the pavement.

6.12.2 EMBANKMENT REINFORCEMENT

Embankments are constructed using a wide range of soil materials. Geosynthetic reinforcement improves the following:

- Increases tensile strength of fill material.
- Increases FOS or enables us to provide steeper slopes.

For embankments more than 50' (15 m) high, a stability analysis should be performed. Reinforcement may be needed to satisfy stability requirements. This may also be necessary with lower strength soils, or steeper slopes, even when the embankment is not so tall.

Table 6.6 Recommended Factor of Safety (FOS) for Geotechnical Analyses

Type of Structure	F.O.S.
Slope Stability	
Cut	1.50
Fill	1.25
Global Stability	
Slope failure (Embankment)	1.25
Tie Back Pull-Out for Drilled Pier	2.00

Table No. 6.7 External Stability Resistance Factors for MSE Walls

STABILITY MODE	CONDITIONS	RESISTANCE FACTOR
Bearing Resistance		0.65
Sliding Resistance		1.00
Overall (Global) Stability	Where geotechnical parameters are well defined, and the slope does not support or contain a structural element	0.75
	Where geotechnical parameters are based on limited information, or the slope contains or supports a structural element	0.65

(AASHTO TABLE 11.5.6-1)

Note: For other systems not covered here, use FHWA and AASHTO LRFD guidelines.