GENERAL INSTRUCTIONS FOR BRIDGE STRUCTURE INVESTIGATIONS

JULY 1996

GEOTECHNICAL SECTION
DIVISION OF MATERIALS AND TESTS
INDIANA DEPARTMENT OF TRANSPORTATION

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INDIANA DEPARTMENT OF TRANSPORTATION
STATE OF INDIANA

GENERAL INSTRUCTIONS FOR BRIDGE INVESTIGATIONS

1. INTRODUCTION

The General Instructions for Bridge Structure Investigations are guidelines which will be followed except in special situations as determined by the INDOT Chief Geotechnical Engineer. Equipment, methods of boring and sampling, testing if required, reports and other details for structure borings shall be as specified in the Agreement or Contract (i.e., Exhibit "C"). Split-spoon sample borings shall be done according to AASHTO T-206. These instructions shall pertain to the locations and depth of structure borings and the pertinent calculations to be used in regard to pile design, scour depth, settlement, negative skin friction, lateral squeeze, lateral pile loading, and seismic activity considerations.

2. GENERAL GUIDELINES FOR BORINGS

2.1 Location of Borings

The INDOT Division of Design or the Design Consultant shall furnish plans of the structures for which borings are to be made. Generally, the plans shall consist of road plan and profile sheets, a situation plan showing the location of substructure elements and cross-sections of the structure's approaches. The plan and profile sheets will have included on them the maximum high water elevation and the stream bed elevation. In general, there shall be a boring made near one end of each pier and end bent with the borings alternating right and left of the centerline of the structures. Twin structures shall be considered as separate structures. Additional borings shall be required as described in the following sections, or as directed by the Engineer. In the case of skewed structures, the borings should be located at the extreme end of the end bents to better determine any subsurface variation at the maximum end limits of such proposed structure.

2.2 Depth of Borings

The following general guidelines have been established for a working load up to 625 kN (70 ton) on a single pile (except for H-piles, see Exhibit "C"). The first boring performed should be the one for an interior pier and shall be drilled a minimum depth of 28 m (92 ft.) below ground elevation, unless bedrock is encountered at a shallower depth. The remaining borings shall generally penetrate to an approximate depth of 15 m (50 ft.) below the ground elevation. However, if higher loading piles are proposed, deeper borings shall be performed. To determine the depths, engineering judgement shall be used based on the loading condition. The deeper boring shall always be drilled first at each structure, and should be located so as to gain the most information, such as at the lowest elevation, or in the flowline.

For box culverts wider than 3 m (10 ft.), the minimum depth of boring shall be 9 m (30 ft.). The last 6 m (20 ft.) of the boring should have a blow count of 15 or more.

In the case of stream crossings, the boring depths shall penetrate a minimum of 4.5 m (15 ft.) below the maximum actual scour depth or to a depth below the maximum actual scour depth sufficient to carry the pile loads with the scourable overburden materials removed, whichever is greater. The latter depth shall extend 1.5 m (5 ft.) below the anticipated pile tip elevations. Engineering judgement shall be required to establish the pile tip elevations required to carry the pile loads and should be handled on an individual basis for each structure. Specific guidelines for the final depth of borings in soil and in bedrock are outlined below.

2.2.1 Borings in Soil

Borings in any soils shall penetrate a minimum of 6 m (20 ft.) continuous into material having a standard penetration value of fifteen (15) or greater. If this minimum depth of penetration of fifteen (15) or more blow material has not been obtained at the proposed boring depth, the boring shall be extended until this requirement is met. Borings shall be extended such that the soils encountered within the borings will be capable of supporting the anticipated loads. The depths of boring shall extend a minimum of 1.5 m (5 ft.) below the anticipated tip elevations.

When ground water is encountered, water should be added to the hole to maintain the water level in the hole at, or above, the ground water level, to aid in avoiding a quick condition when granular soils are encountered. This precaution will keep the sand from coming up into the casing. For loose to medium dense sand below the water table, the bore hole may need to be stabilized with drilling fluid to prevent heave of the sand up into the casing. The ball check valve in the split-spoon sampler should not be removed, and washing through the spoon will not be permitted.

2.2.2 Borings With Rock Coring

In general, if rock is encountered in the borings shown on the plans, rock coring will be required in each boring. Rock coring should not begin until auger refusal is obtained. RQD (Rock Quality Designation) values should be calculated and recorded before transporting the core sample from the boring location. Rock coring should not begin in weathered shale, weathered limestone, etc. unless absolutely necessary. Coring and sampling should not stop in coal.

If rock is encountered at a depth of 0.0 to 4.5 m (0 to 15 ft.) below the stream bed elevation, all assigned borings shall penetrate to rock. Rock coring for each of the above borings shall penetrate a minimum of 1.5 m (5 ft.) into rock having seventy-five percent (75%) recovery, or to an approximate maximum of 4.5 m (15 ft.) below the stream bed elevation. In the case of a grade separation structure, if rock is encountered at a depth of 0.0 to 3 m (0 to 10 ft.) below the proposed footing elevation, the core boring shall penetrate a minimum of 1.5 m (5 ft.) into rock

having seventy-five percent (75%) recovery or to an approximate maximum of 3 m (10 ft.) below the proposed footing elevation.

If shallow rock is encountered and spread footings are anticipated on rock, coring a minimum of 3 m (10 ft.) into the rock will be required for the borings. Also, if rock elevations vary and spread footings are anticipated on rock, all rock soundings shall be cored a minimum of 1.5 m (5 ft.) into rock. If there are layers of soft materials or voids in the cored rock, coring a minimum of 3 m (10 ft.) into the sound rock will be required below the soft layer or voids.

If rock is encountered below the interior pier footings at a depth of 4.5 m to 9 m (15 to 30 ft.) below the stream bed elevation, or 3 m to 9 m (10 to 30 ft.) below the footing elevation in the case of grade separation, the interior borings shall be cored and penetrate a minimum of 1.5 m (5 ft.) into rock having a recovery of approximately seventy-five percent (75%) or to a maximum of 3 m (10 ft.) below the rock surface. The borings for the end bents shall be made by split spoon sampling and extend to rock, unless the blow counts indicate that the depth is sufficient to carry the pile loads as previously specified.

If rock is encountered at a depth between 9 m to 15 m (30 to 50 ft.) in the 15 m (50 ft.) hole, which is to be the first hole bored as specified previously in this section, the core boring shall penetrate a minimum of 1.5 m (5 ft.) into rock having a recovery of approximately fifty percent (50%) or to a maximum of 3 m (10 ft.) below the bedrock surface. The other borings shown on the plans shall be made by split spoon sampling and extend to the rock, unless the blow counts indicate that a sounding type boring would be adequate below 9 m (30 ft.) or the approximate depth shown on the plans to carry the pile loads.

If an end bearing pile foundation on rock is indicated, sounding type auger borings shall be required if rock elevations in all the borings are not within 0.6 m (2 ft.) of one another. The soundings shall be made at the opposite end from the boring made previously for each pier and each end bent. These soundings shall extend to auger refusal in bedrock.

3. STATIC DESIGN PROCEDURE TO PREDICT PILE CAPACITIES

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 Geotechnical Section is recommending that all Geotechnical Consultants review the methods, assumptions and values used by the INDOT Geotechnical Section to compute the ultimate bearing capacity for piles. The Geotechnical Consultants should analyze both steel encased concrete piles and steel H-piles. The following

approach for calculating the ultimate bearing capacity will be used in checking the ultimate bearing capacities computed by INDOT's Geotechnical Consultants.

The pile capacity should be determined using the computer program SPILE 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. A factor of safety of 2.5 should be used to calculate the pile capacity with these methods.

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

$$Q_{ult} = Q_s + Q_p \qquad \text{(Equation 1)}$$

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.

3.1 Soils with Frictional Strength

3.1.1 Skin Resistance

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

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

$$Q_s = \sum_{0}^{D} K_\delta\, C_F\, P_d\, \frac{\sin(\omega + \delta)}{\cos \omega}\, C_d\, \Delta d \qquad \text{(Equation 2)}$$

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

$$Q_s = \sum_{O}^{D} K_\delta\, C_F\, P_d\, \sin \delta\, C_d\, \Delta d$$

Where:

Q_s =	Total skin friction capacity
K =	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

$\Delta d =$ Depth increment below ground surface
$C_F =$ Correction factor for K_δ when $\delta \neq \phi$ (soil friction angle)

To avoid numerical intergration, 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 (3) becomes:

$$q_s = K_\delta \, C_F \, P_d \sin \delta \, C_d \, D \qquad \text{(Equation 4)}$$

Where within the segment selected:

P_d = The average effective overburden pressure in segment D
C_d = The average pile perimeter
D = The segment length
q_s = The 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 soils, i. e., normal force times tangent of friction angle, N Tanϕ. 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 ϕ, shearing resistance against the sides of the pile.

<u>Computational Steps for Non-Tapered Piles</u>

a. Draw the existing effective overburden pressure (P_o) diagram.

b. Choose a trial pile length.

c. Subdivide the pile according to changes in the unit weight or soil friction angle (ϕ).

d. Compute the average volume per foot of each segment.

e. Enter Figure 4 with that volume and the pile type to determine δ/ϕ and compute δ.

f. Enter the appropriate chart(s) in Figure 5-8 to determine K for ϕ.

g. If $\delta \neq \phi$, enter Figure 9 with ϕ and δ/ϕ, to determine a correction factor C_F to be applied to K_δ.

h. Determine the average values of effective overburden pressure and pile perimeter for each pile segment.

I. Compute q_s from equation 4 for all pile segments and sum to find the ultimate frictional resistance developed by the pile.

For tapered piles Figure 5-8 must be entered with both ϕ and ω to determine K_δ. Also, equation 2 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^o for soft or rounded particles and 36^o for hard angular deposits. This method also tends to overpredict capacity for piles larger that 600 mm (24 inches) in nominal width. The angle of internal friction for cohesionless soils should be limited to a maximum of 35^o in the SPILE program.

3.1.2 End Bearing Capacity

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

$$Q_p = A_p \propto P_d N'_q \qquad \text{(Equation 5)}$$

Where:

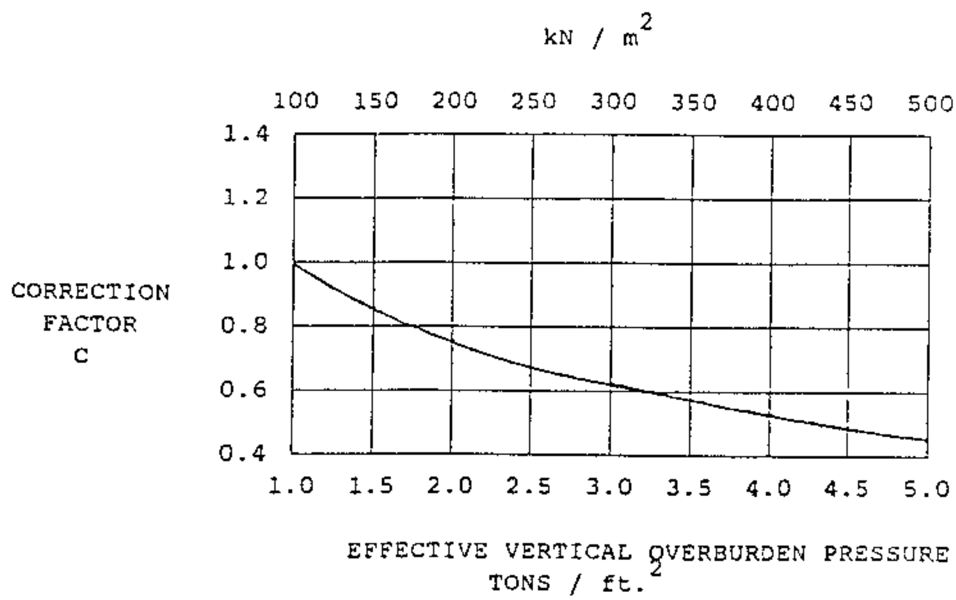
Q_p = end bearing capacity
A_p = pile end area
$\propto$ = dimensionless factor dependent on depth-width relationship (see Figure 10)
P_d = effective overburden pressure at the pile point
N'_q = bearing capacity factor from Figure 10

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 150 kN/m^2 (3000 psf). More recently, the authors have suggested that further limitation 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 in Figure 11. The end bearing capacity should be taken as the lesser of Q_p or Q_{LIM}.

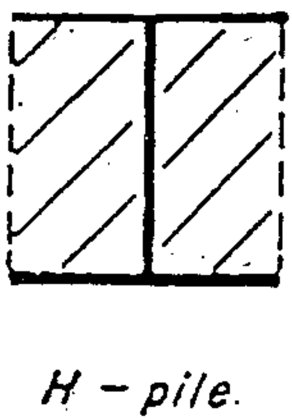
The actual steel area should be used to calculate the point resistance in the cohesionless soils.

3.1.3 Ultimate Pile Capacity

The ultimate pile capacity is the sum of all q_s values and Q_p. However, for foundation design sum only 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.



<u>FIGURE 1.</u> CHART FOR CORRECTION OF N-VALUES IN SAND FOR INFLUENCE OF OVERBURDEN PRESSURE--REFERENCE VALUE OF EFFECTIVE OVERBURDEN PRESSURE OF 100 KN/m^2 (1.0 ton/sq. ft.) (MODIFIED FROM PECK, et.al., 1979)



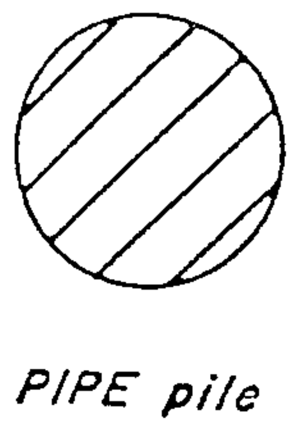


FIGURE 2. SUGGESTED END AREAS FOR DRIVEN H AND PIPE PILES WHERE PLUG WILL FORM.

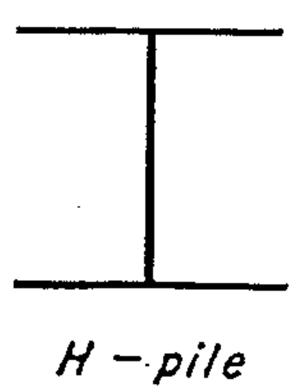
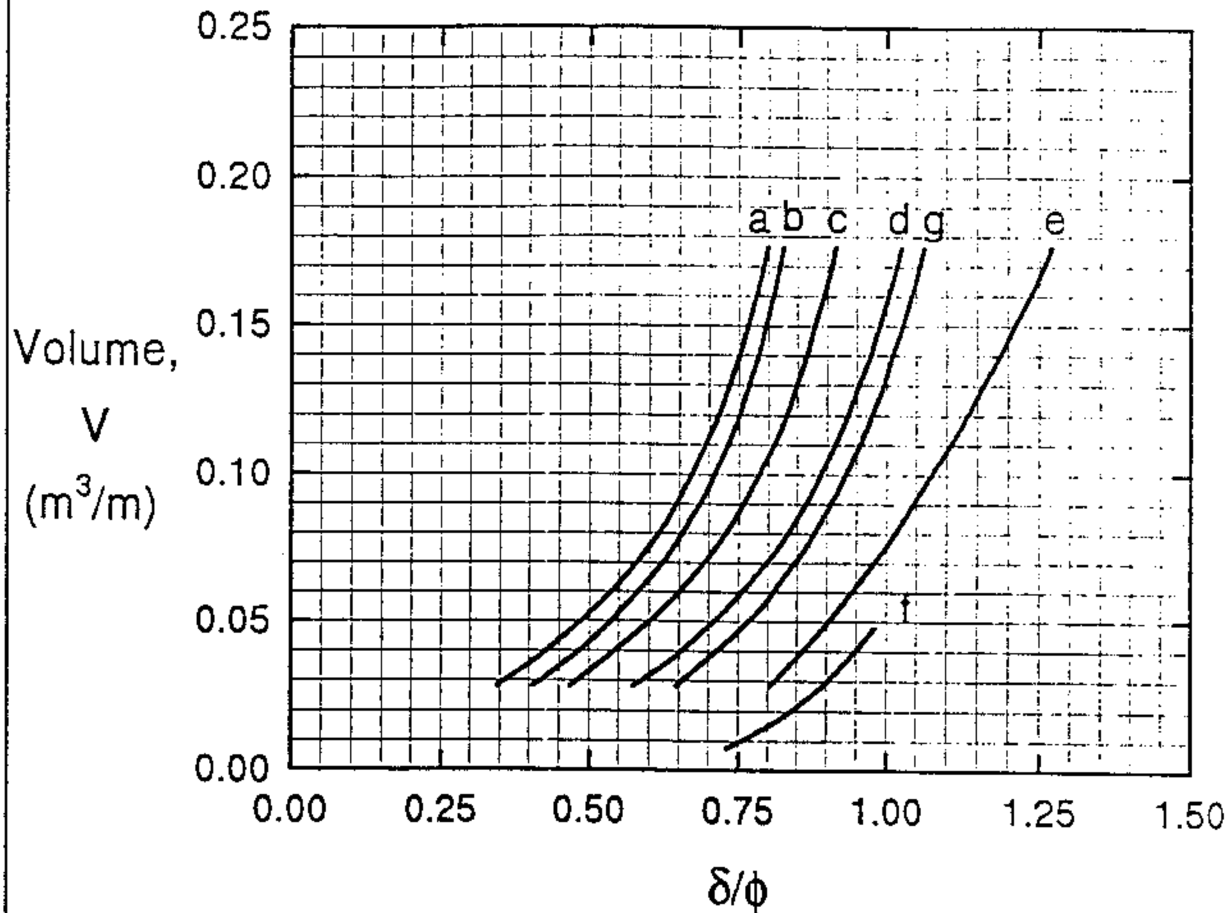


FIGURE 3. SUGGESTED END AREA FOR DRIVEN H-PILE WHERE PLUG WILL NOT FORM.



a. Closed end pipe and non-tapered portion of monotube piles
b. Timber piles
c. Precast concrete piles
d. Raymond step-taper piles
e. Raymond uniform taper piles
f. H-piles
g. Tapered portion of monotube piles

FIGURE 4 RELATION OF δ/ϕ AND PILE DISPLACEMENT, V, FOR VARIOUS TYPES OF PILES

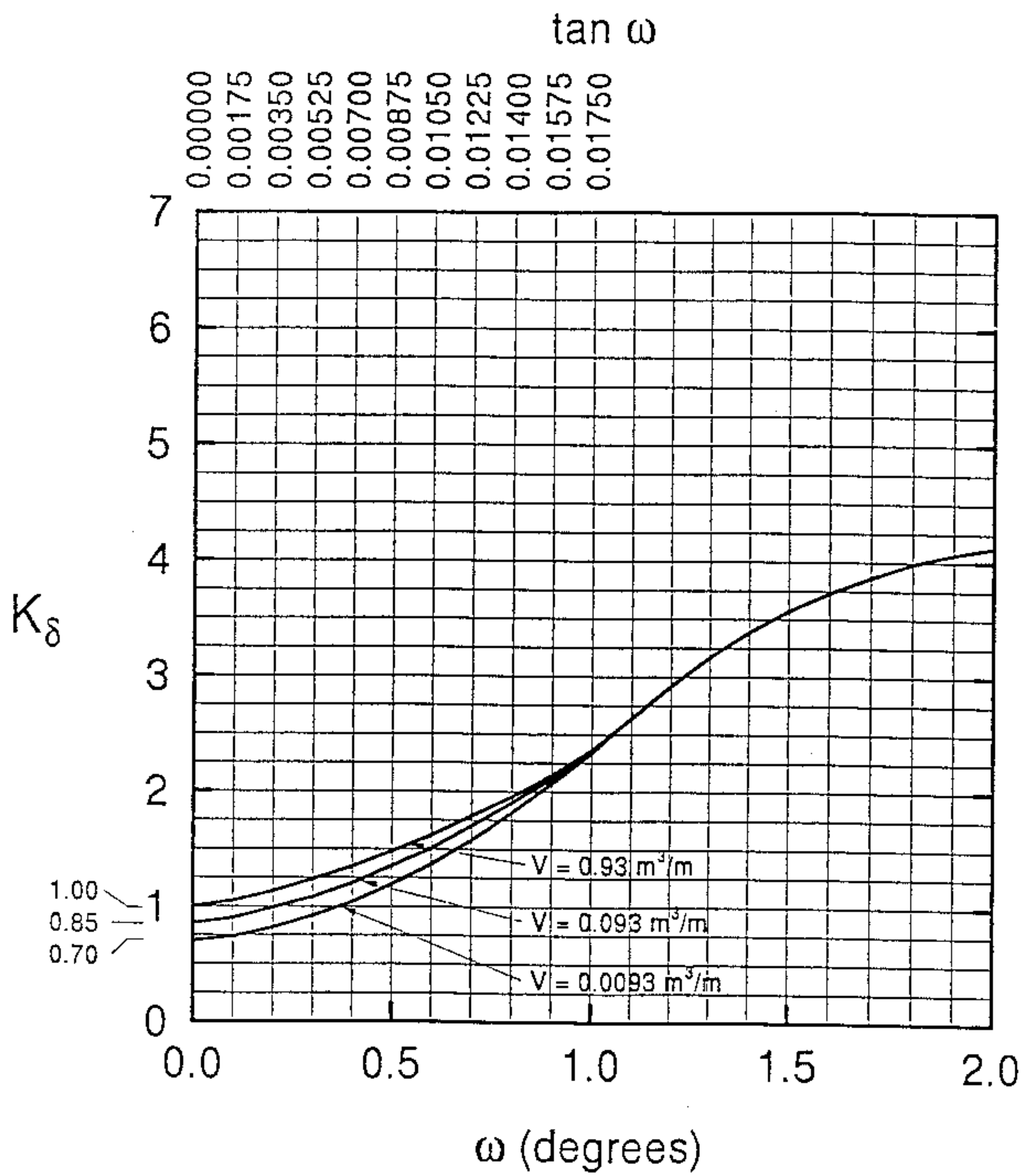


FIGURE 5 DESIGN CURVES FOR EVALUATING K_δ FOR PILES WHEN $\phi = 25°$ (AFTER NORDLUND 1979)

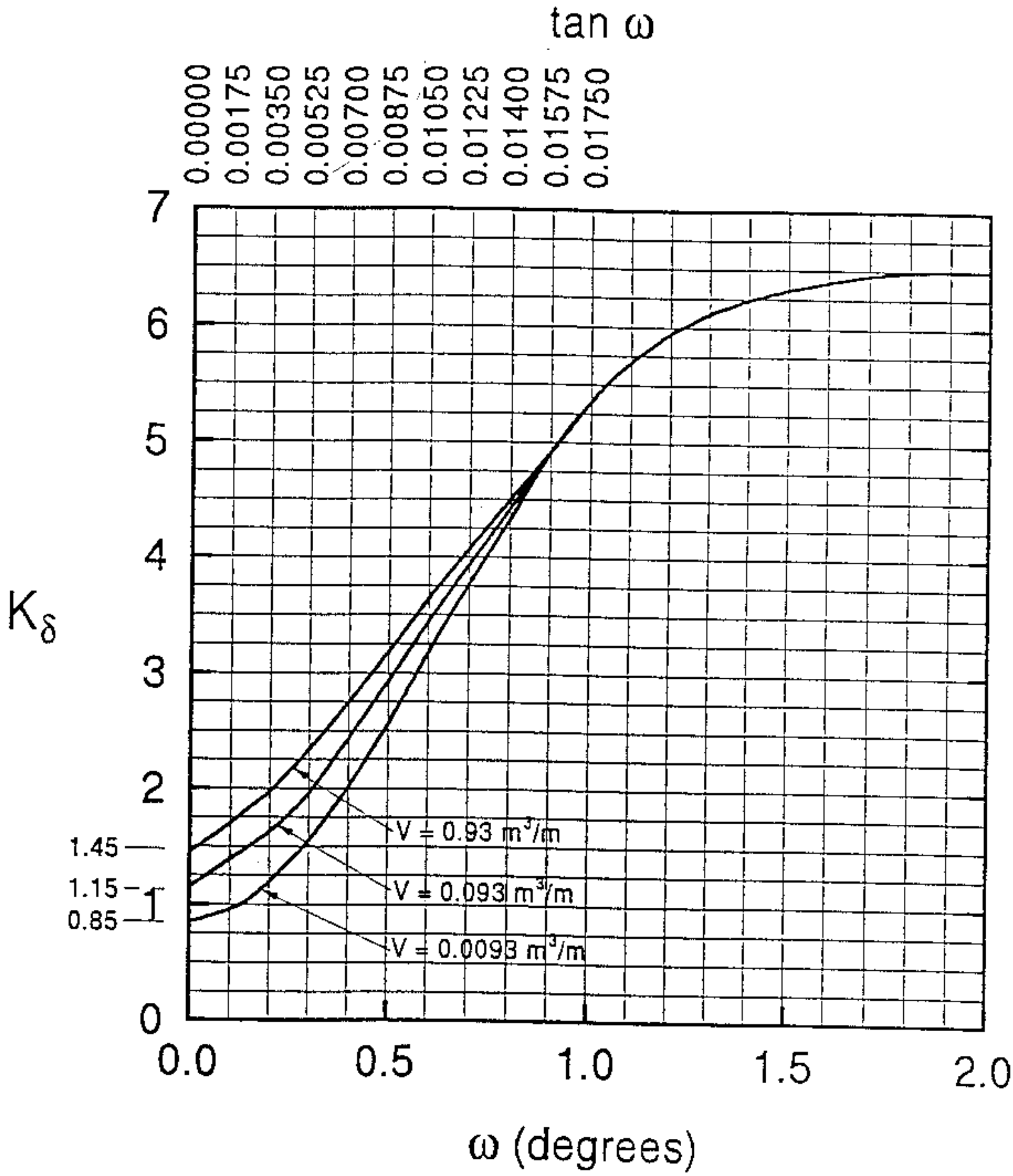


FIGURE 6 DESIGN CURVES FOR EVALUATING K_δ FOR PILES WHEN $\phi = 30^\circ$ (AFTER NORDLUND 1979)

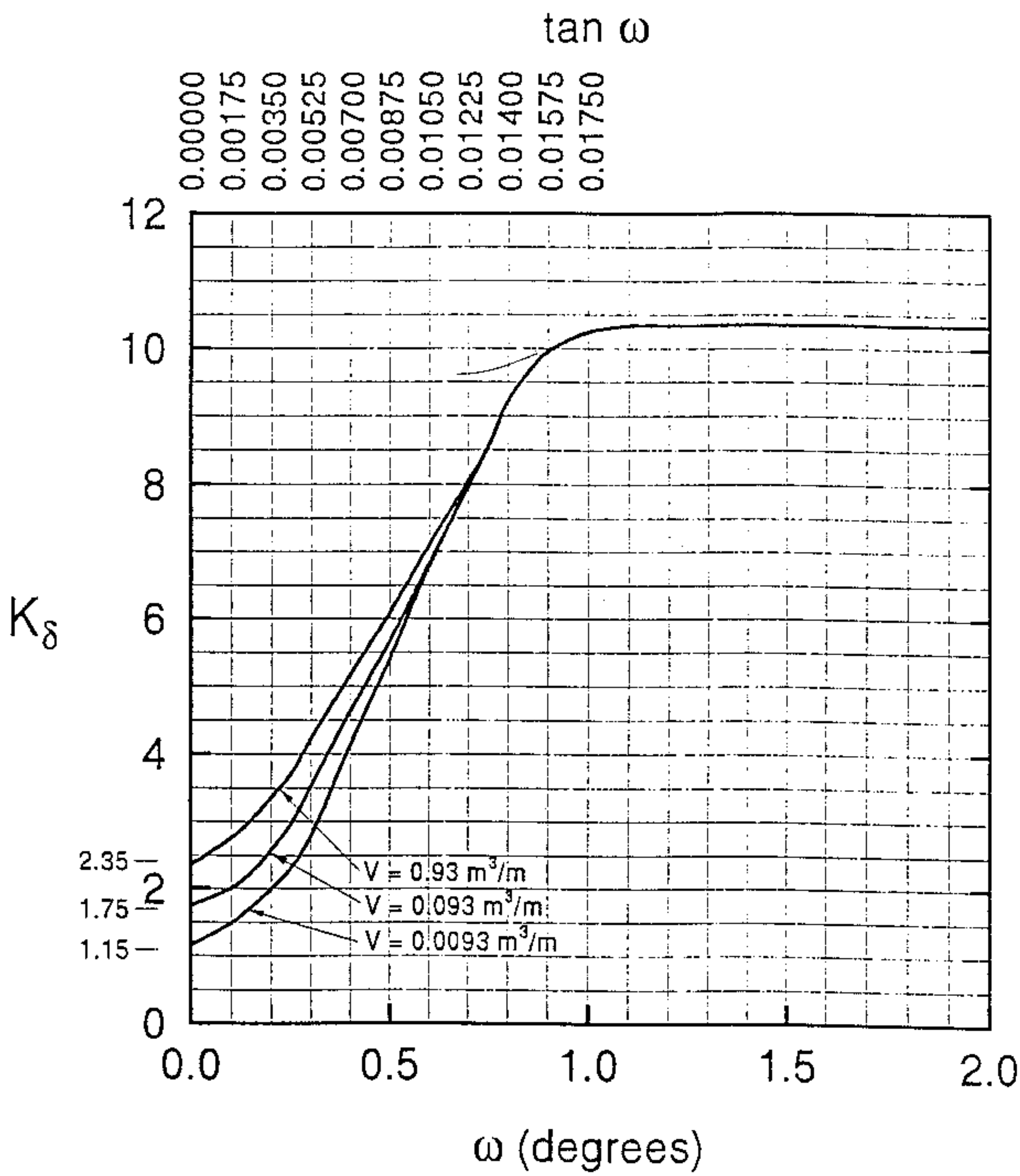


FIGURE 7 DESIGN CURVES FOR EVALUATING K_{δ} FOR PILES WHEN ϕ = 35° (AFTER NORDLUND 1979)

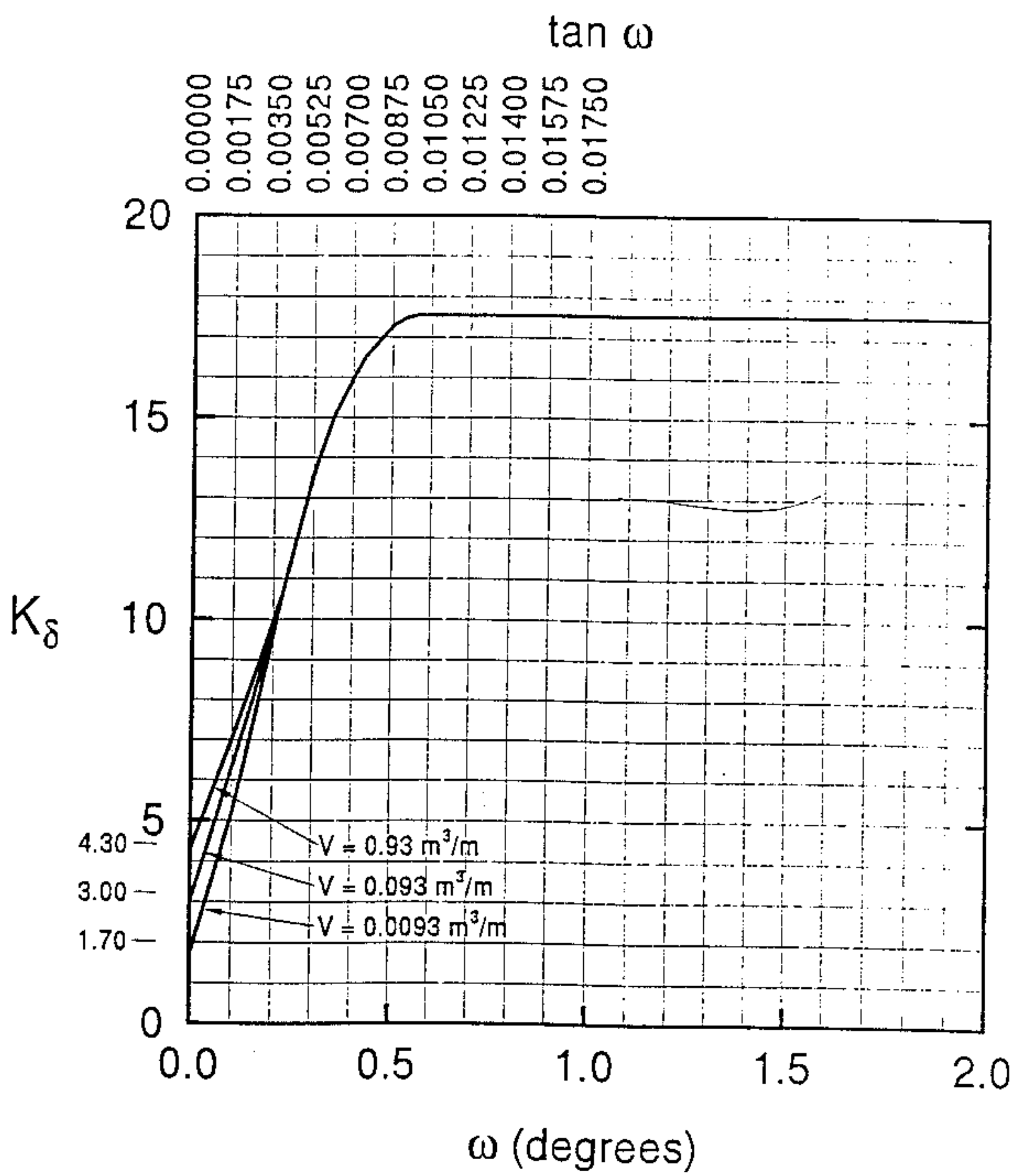


FIGURE 8 DESIGN CURVES FOR EVALUATING K_δ FOR PILES WHEN ϕ = 40° (AFTER NORDLUND 1979)

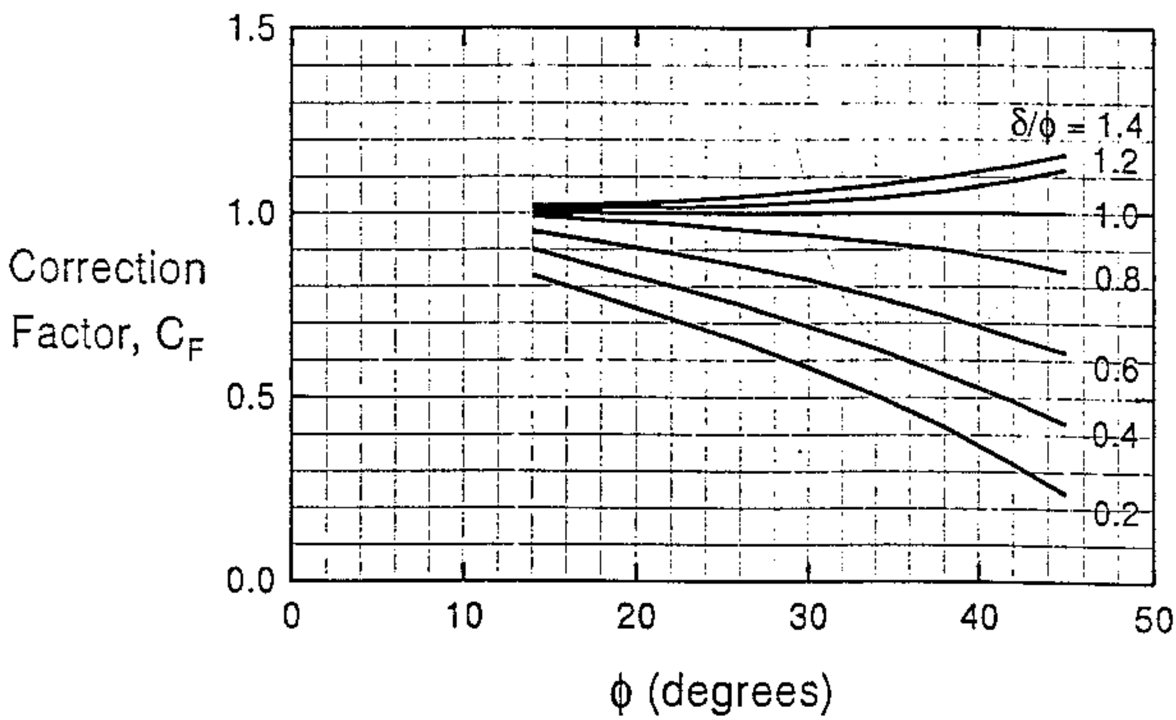
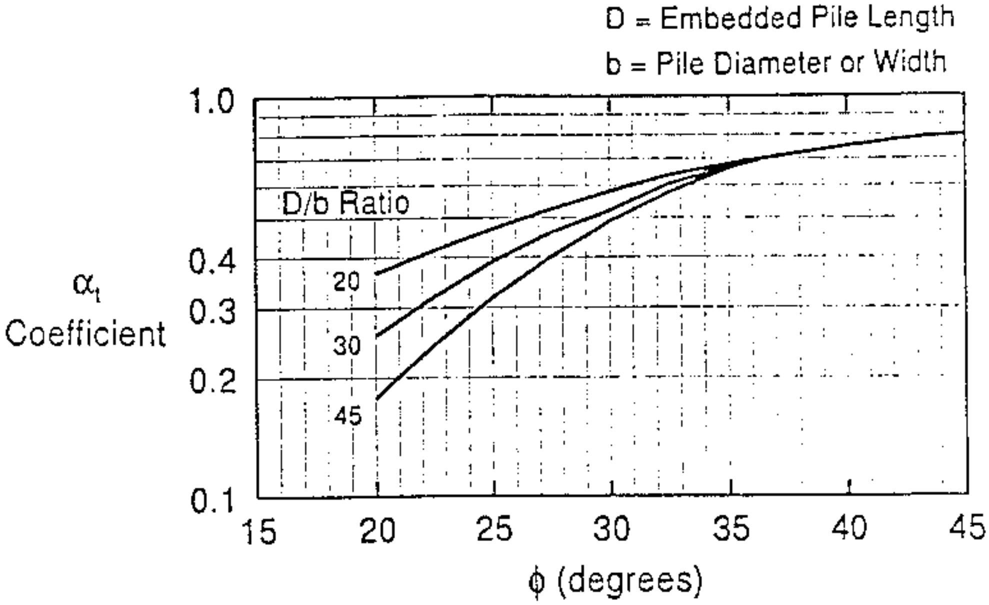


FIGURE 9 CORRECTION FACTOR FOR K_δ WHEN $\delta \neq \phi$



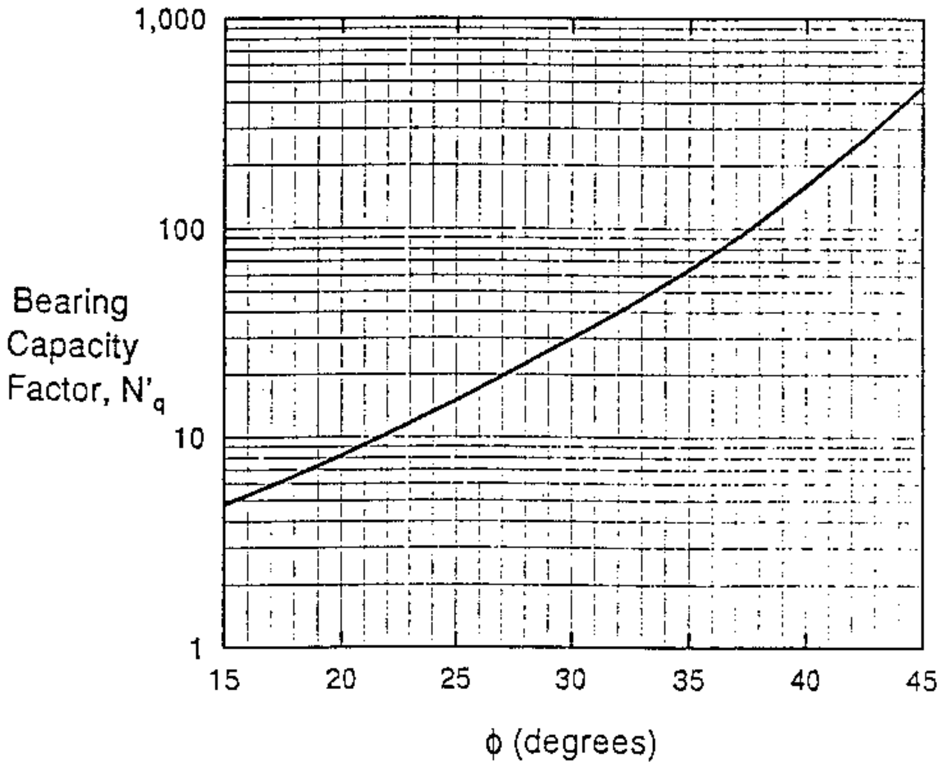


FIGURE 10 DETERMINATION OF α COEFFICIENT AND VARIATION OF BEARING CAPACITY FACTORS WITH Ø

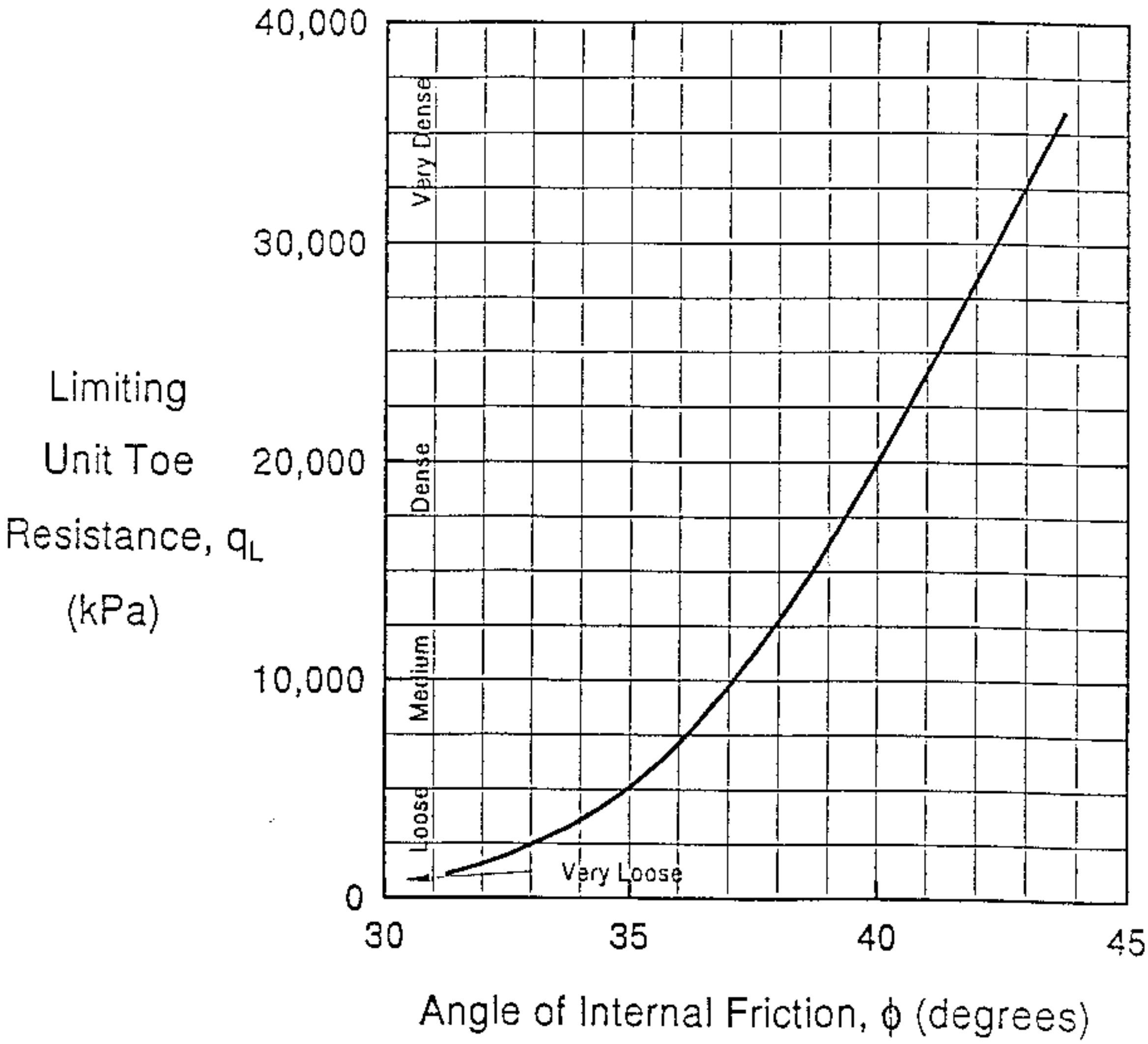


FIGURE 11. RELATIONSHIP BETWEEN MAXIMUM UNIT PILE POINT RESISTANCE AND FRICTION ANGLE FOR COHESIONLESS SOILS (AFTER MEYERHOF, 1976)

3.2 Clay and Cohesive Soils

The ultimate capacity of a pile (Q_{ult}), in clay can be determined by summing the total frictional resistance (Q_{sf}) and the maximum end bearing resistance (Q_{eb}) as previously stated in Equation 1 for non-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)}.

3.2.1 Skin Friction Resistance

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:

$$Q_{sf} = f_s \, P \, L \qquad \text{(Equation 6)}$$

where:

- f_s = average unit skin friction or adhesion in KN/m^2 (tsf)
- P = perimeter of the pile in meter (ft.)
- L = embedded length of the pile in meter (ft.)

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

$$f_s = c_a = \propto c_u \qquad \text{(Equation 7)}$$

In this application, $\propto$ 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 "$\propto$ Method".

The coefficient $\propto$ depends on the nature and strength of the clay, pile dimension, method of pile installation and time effects. The values of $\propto$ vary within wide limits and decrease rapidly with increasing shear strength. The values of $\propto$ can be obtained from Figure 12.

In the case of H-piles, there is an uncertainty as to the development of skin friction along the web and the possibility that the intermixing effects between a soft clay upper layer and a stiffer clay may cause a significant reduction in skin friction. For these reasons, the skin friction for H-piles should be calculated conservatively on <u>a perimeter equal to twice the flange width.</u>

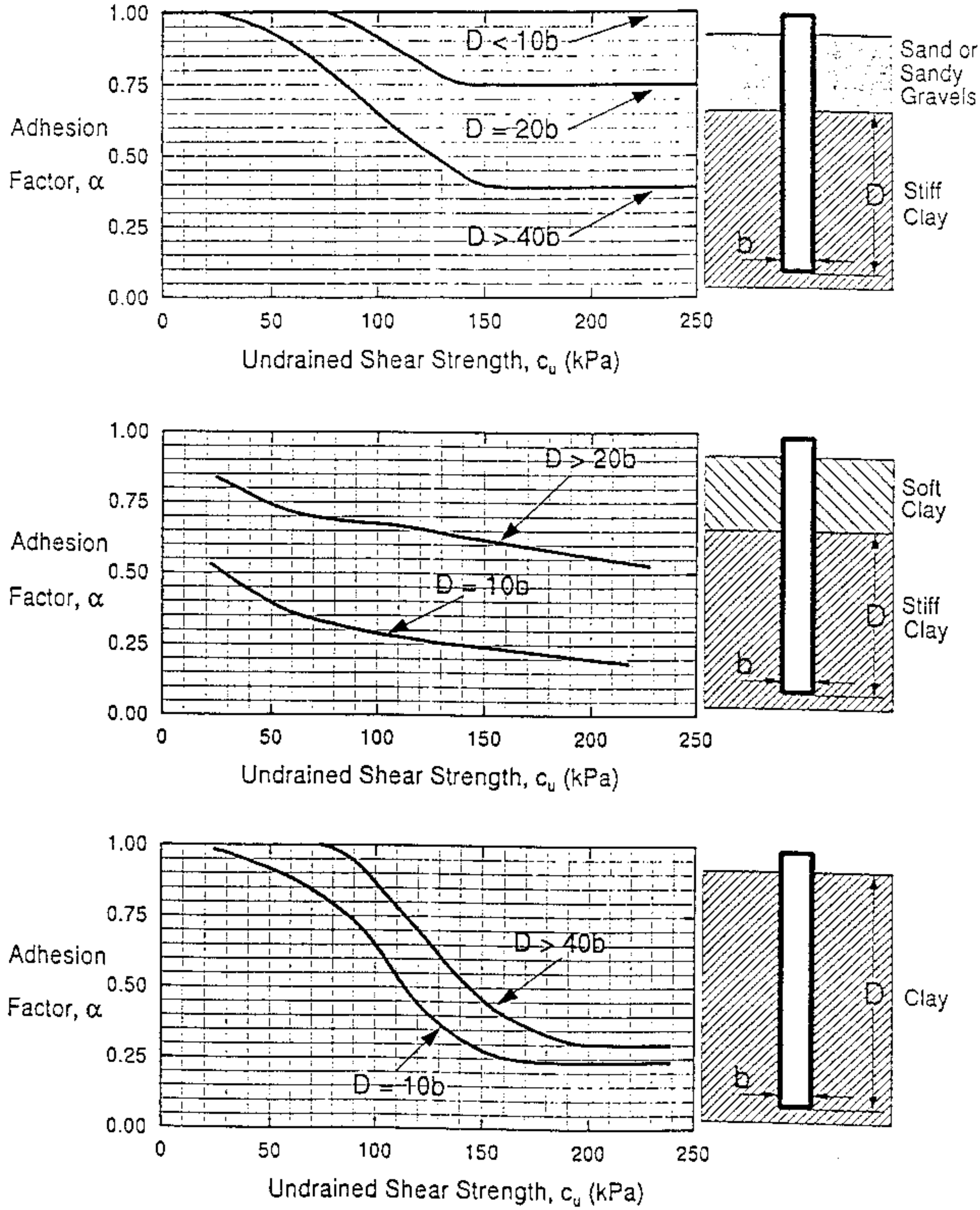


FIGURE 12. ADHESION FACTORS FOR DRIVEN PILES IN CLAY--THE METHOD (AFTER TOMLINSON, 1980).

Determining Skin Friction Resistance using The "∝ Method"

Step 1: Determine adhesion factor ∝ from Figure 12.

Enter Figure 12 with pile length in clay and undrained shear strength of soil (cu) in KN/m^2 (psf). Use appropriate curves for situations (a), (b), or (c) shown in the figure.

Step 2: Compute ultimate unit skin friction resistance (f_s)

$f_s = C_a$ (adhesion) = (∝) x (C_u)

Step 3: Compute total ultimate skin friction resistance

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

where: A_s = Pile surface area

3.2.2 End (Point) Bearing Capacity

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

$$Q_{eb} = q_{eb}(A_t) = (C\ Nc + P_v\ Nq + 1/2\ \bar{(}\ D\ N_{(})\ A_t \qquad \text{(Equation 8)}$$

where:

q_{eb} = ultimate tip bearing capacity

A_t = area of pile tip

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

$\bar{(}$ = effective unit soil weight on the vicinity of the tip

P_v = effective vertical stress (limiting overburden of 10-15 D)
D = pile diameter or width

Nc, Nq, $N_{(}$ = deep foundation bearing capacity factors (see Figure 13).

NOTE: since D is usually small, the $N_{(}$ term is often neglected

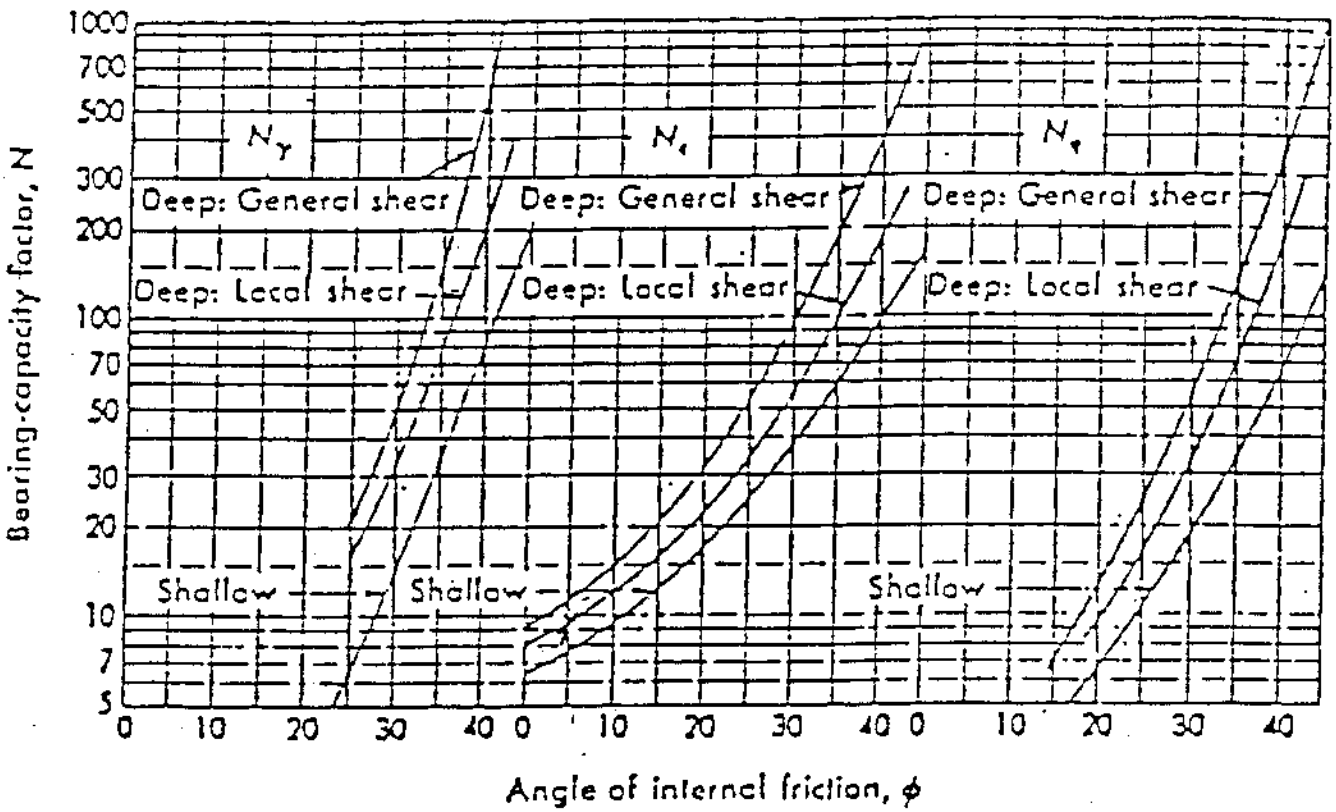
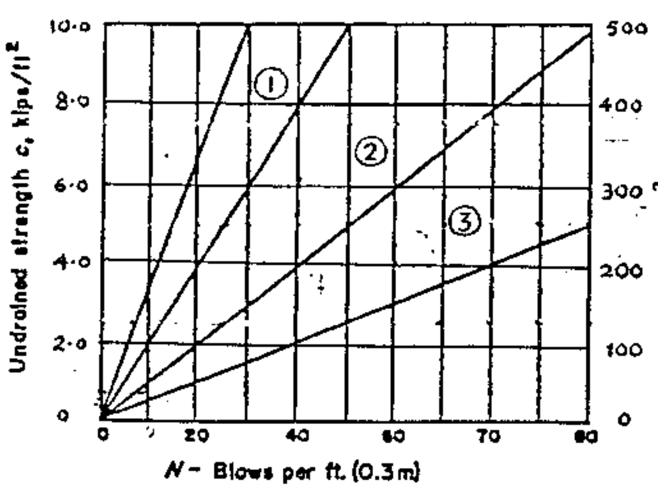


FIGURE 13. BEARING CAPACITY FACTORS FOR SHALLOW AND DEEP SQUARE OR CYLINDRICAL FOUNDATIONS.



Soil Groups

	MOST PROBABLE	POSSIBLE
1	A-7-6	A-7-5
2	A-6, A-7-6	A-4
3	A-2-6, A-2-7	A-2-4, A-6 A-4, A-7-6
	A-4, A-5	A-6, A-7-5

NOTE: an appropriate N_c value should be chosen from Fig. 5 based on a $\phi \geq 0$ for these soils

FIGURE 14. RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH (c) AND PENETRATION RESISTANCE(N) (MODIFIED AFTER SOWERES, 1979).

For clay soils (ϕ = 0, and assume N_q = 0), the end bearing becomes:

$$Q_{eb} = c\ Nc\ A_t \qquad \text{(Equation 9)}$$

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 correlates the penetration resistance (N) to the undrained shear strength (c) based on textural classification. These are useful correlations 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.

3.3 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 (Nc & C) for glacial till should be large so that the ultimate bearing capacity for driven piles will be obtained in the upper portion of the till. Piles should be driven only a few meters (feet) into glacial "hardpan". If the till is predominately non-cohesive, Nordlund's end bearing formula should be used.

3.4 Additional Considerations

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

1. Capacity can change with time.

2. Load transfer can change with time from such causes as creep induced by new fill, lowering the groundwater table, remolding of clay, etc.

3. Settlement of pile, etc.

4. Application to capacity and settlement of pile group

5. Negative skin friction, which is a bearing capacity problem induced by settlement. Some causes are:

a. Placement of clay fill over sand where the fill drags the pile down during consolidation and lateral stresses also increase in sand.

b. Placement of fill over compressible clay where fill causes downdrag and clay also causes downdrag due to consolidation effects.

c. 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 ultimate point resistance will be reduced. The ultimate point resistance in the bearing stratum will be governed by the resistance to punching of the pile into the underlying weak soil.

3.5 Piles on Rock

Approximately one-half of the area of Indiana has sedimentary rock near the ground surface {within 15 m (50 feet) or less}. Deep foundations on rock are common where the unconsolidated materials are inadequate to support the ultimate load of the structure. The items listed below should be considered for exploration and design for rock foundations.

1. Steel Encased Concrete (S.E.C.) piles should be considered when a deep foundation is to be supported by soil of shale without any rock floaters. Otherwise, H-piles driven to sound rock should be recommended.

2. 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.

3. Pile tips should not be placed above any coal deposits.

4. Rock Quality Designation (R.Q.D.) values can provide a qualitative assessment of rock mass as shown in Table 1. The RQD is computed by summing the length of all pieces of core equal to or longer than 100 mm (4 in.), 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.

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

TABLE 1. Engineering Classification For In-Situ Rock Quality Using The Rock Quality Designation (RQD).

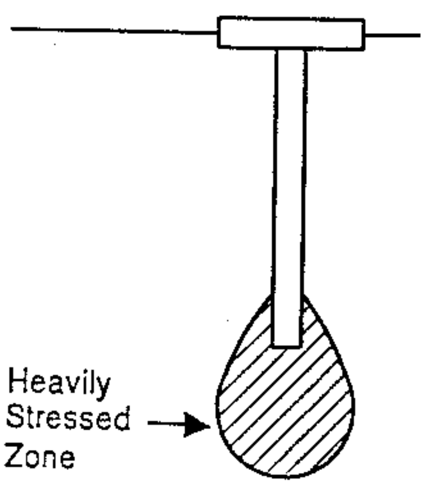
3.6 Scour Depth

The expected scour depth should be considered for every bridge structure over water. The engineer should design the permanent pile capacity to mobilize the required soil resistance below the scour depth. The minimum pile tip elevation will be 3 m (10 ft.) 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 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 Consultant shall send a written request for a scour estimate to the INDOT - Hydraulic Section (Division of Design) along with the copies of the grain size curves. Once the calculated scour depth is received, the Geotechnical Consultant will perform the structure analysis. The scour depth for Q_{100} is generally considered in the structure analysis.

4. PILE GROUP CAPACITY

If pile group capacity analysis is required on a given project, approval must be obtained from the INDOT Geotechnical Section 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 15 for a single pile shows that only a small zone of soil around and below the pile is subjected to vertical stress. Figure 16 for a pile group shows that a considerable depth of soil around and below the 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 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.



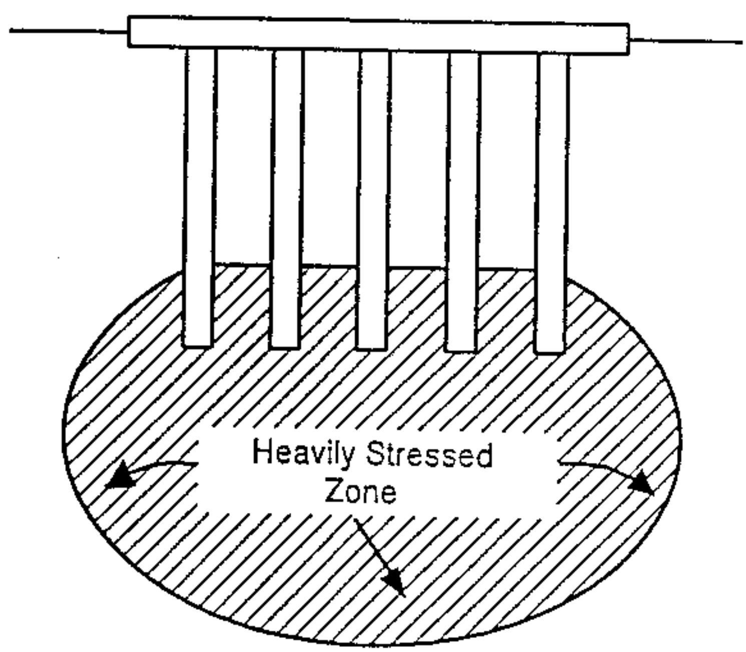


FIGURE 15: STRESSED ZONE UNDER END BEARING SINGLE PILE

FIGURE 16: STRESSED ZONE UNDER END BEARING PILE GROUP

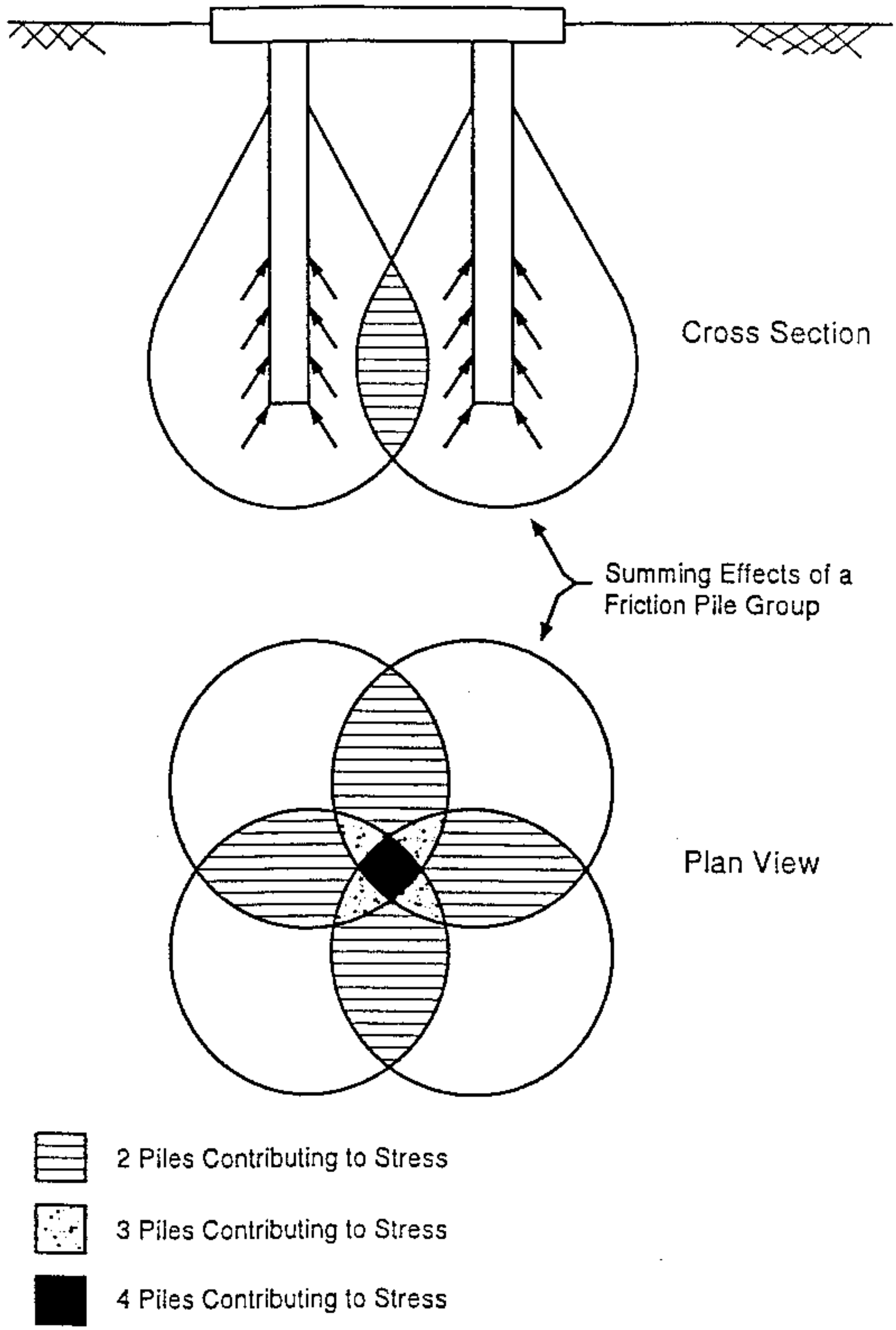


FIGURE 17. OVERLAPPING STRESSED SOIL AREAS FOR A PILE GROUP.

4.1 Pile Group Capacity in Cohesionless Soils

In cohesionless soils, the ultimate group load capacity of driven piles with a center spacing of less than three (3) pile diameters is greater than the sum of the ultimate 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 spacings greater than three (3) times the average pile diameter act as individual piles.

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

1. The ultimate group load in soil not underlain by a weak deposit should be taken as the sum of the single pile capacities.

2. If a group founded in a firm bearing stratum of limited thickness is underlain by a weak deposit, the ultimate group load is given by the smaller value of either:

    a. The sum of the single pile capacities or,

    b. By a 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 (2) pile diameters, which is rarely the case. The method shown for cohesive soils may be used to investigate the possibility of a block failure.

3. Piles in groups should not be installed at spacings less than three (3) times the average pile diameter.

4.2 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:

1. For pile groups driven in clays with undrained shear strengths of less than 100 kN/m^2 (2,000 psf) and for spacings of three (3) times the average pile diameter, the group efficiency can be taken to be equal to seventy percent (70%). If the spacing is greater than four (4) times the average pile diameter, then a group efficiency equal to one-hundred percent (100%) can be used.

2. For pile groups in clays with undrained shear strength in excess of 100 kN/m^2 (2,000 psf), use a group efficiency equal to one-hundred percent (100%).

3. Investigate the possibility of a block failure. Recommended method is described in the next section.

4. Piles should not be installed at spacings less than two (2) times the average pile diameter.

4.3 Ultimate Resistance against Block Failure of Pile Group in Cohesive Soil

A pile group in cohesive soil is shown in Figure 18. An estimate of the ultimate resistance of the pile group against a block failure is provided by the following expression:

$$Qu = (9 \times Cu1 \times B \times L) + (2 \times D \times (B+L) \times Cu2) \qquad \text{(Equation 10)}$$

where:

Qu = Ultimate resistance against block failure

Cu1 = Undrained shear strength of clay below pile tips

Cu2 = Average undrained shear strength of clay around the group

B = Width of group

L = Length of group

D = Length of piles

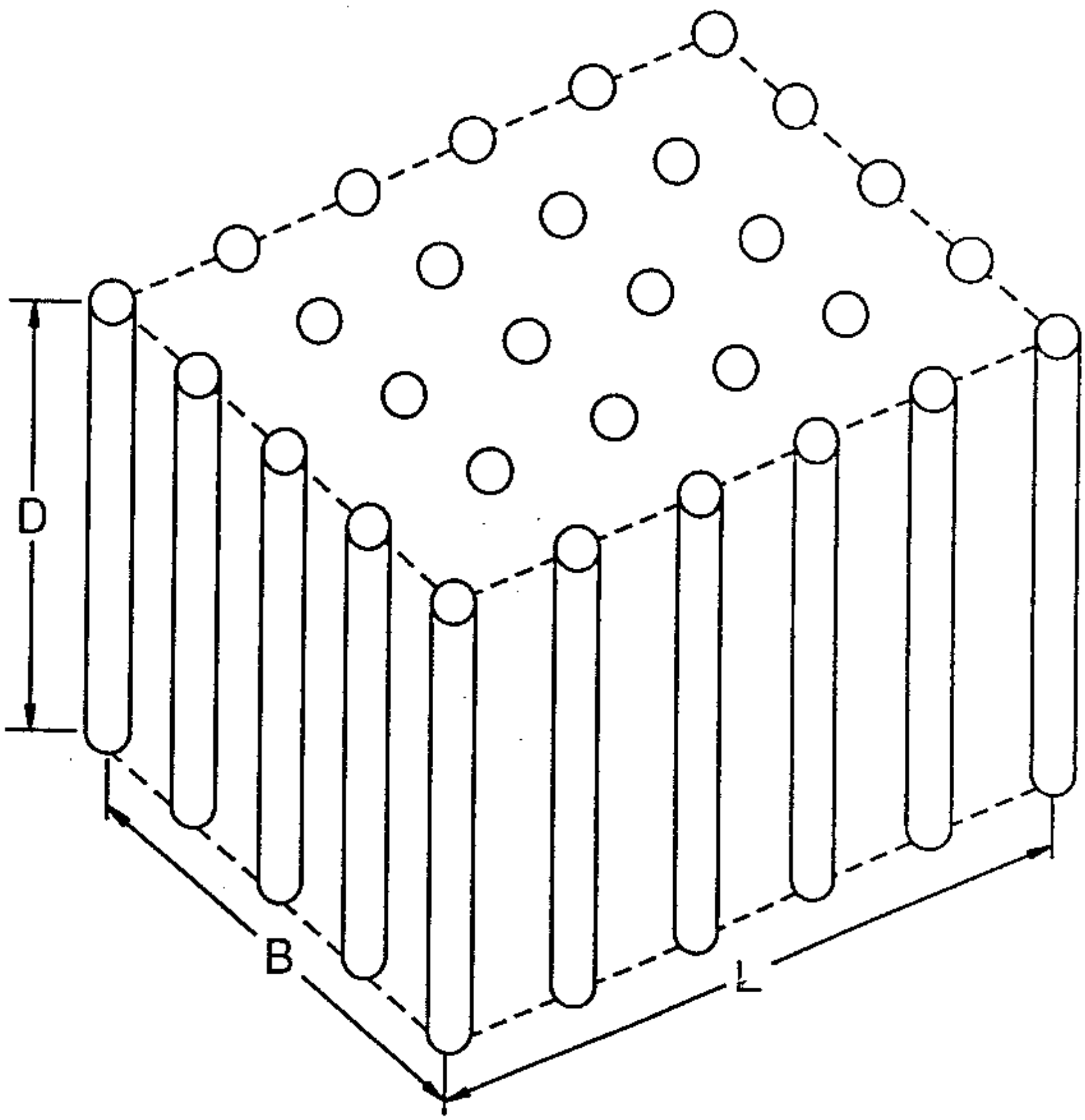


FIGURE 18. PILE GROUP IN COHESIVE SOIL.

4.4 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 overconsolidated 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.

4.4.1 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:

$$\delta = \frac{P \times L}{A \times E} \qquad \text{(Equation 11)}$$

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.

4.4.2 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:

$$S = \frac{2p\,(B)^{1/2}\,I}{N'} \qquad \text{(Equation 11a)(English)}$$

or

$$\frac{95p\,(B)^{1/2}\,I}{N'} \qquad \text{(Equation 11a) (Metric)}$$

For silty sand use the following equation:

$$S = \frac{4p\,(B)^{1/2}\,I}{N'} \qquad \text{(Equation 11b)(English)}$$

or

$$S = \frac{190p\,(B)^{1/2}\,I}{N'} \qquad \text{(Equation 11b) (Metric)}$$

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)

4.4.3 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 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 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).

The following settlement equation is used for computing consolidation settlement:

$$S = H \left[(C_{cr} / (1 + e_o)) \log (p_c / p_o) + ((C_c / (1 + e_o)) \log ((P_o + \Delta P) / P_c) \right] \qquad \text{(Equation 12)}$$

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

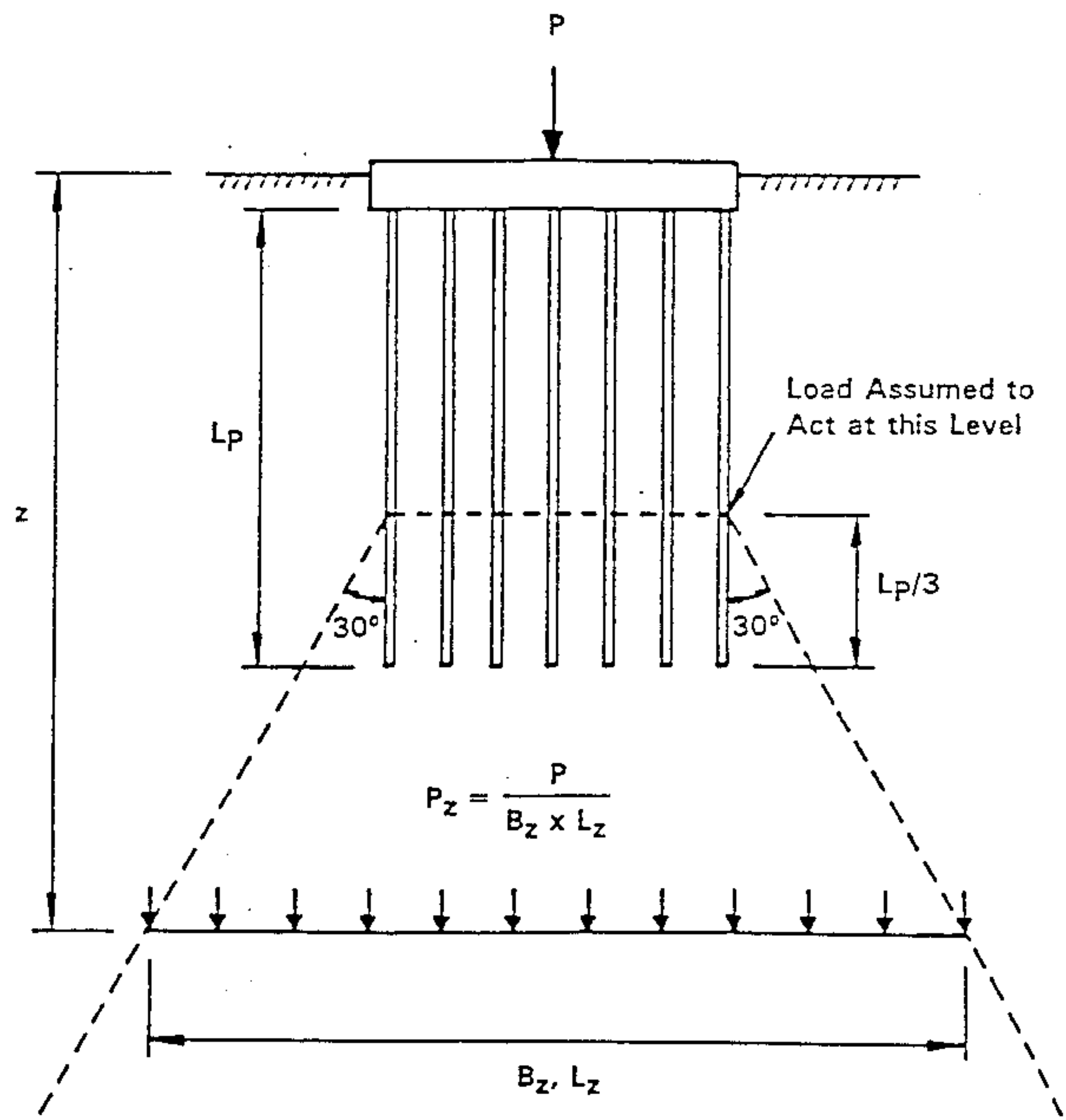


FIGURE 19. STRESS DISTRIBUTION BENEATH PILE GROUP IN CLAY USING THEORETICAL FOOTING CONCEPT (AFTER CANADIAN GEOTECHNICAL SOCIETY, 1978).

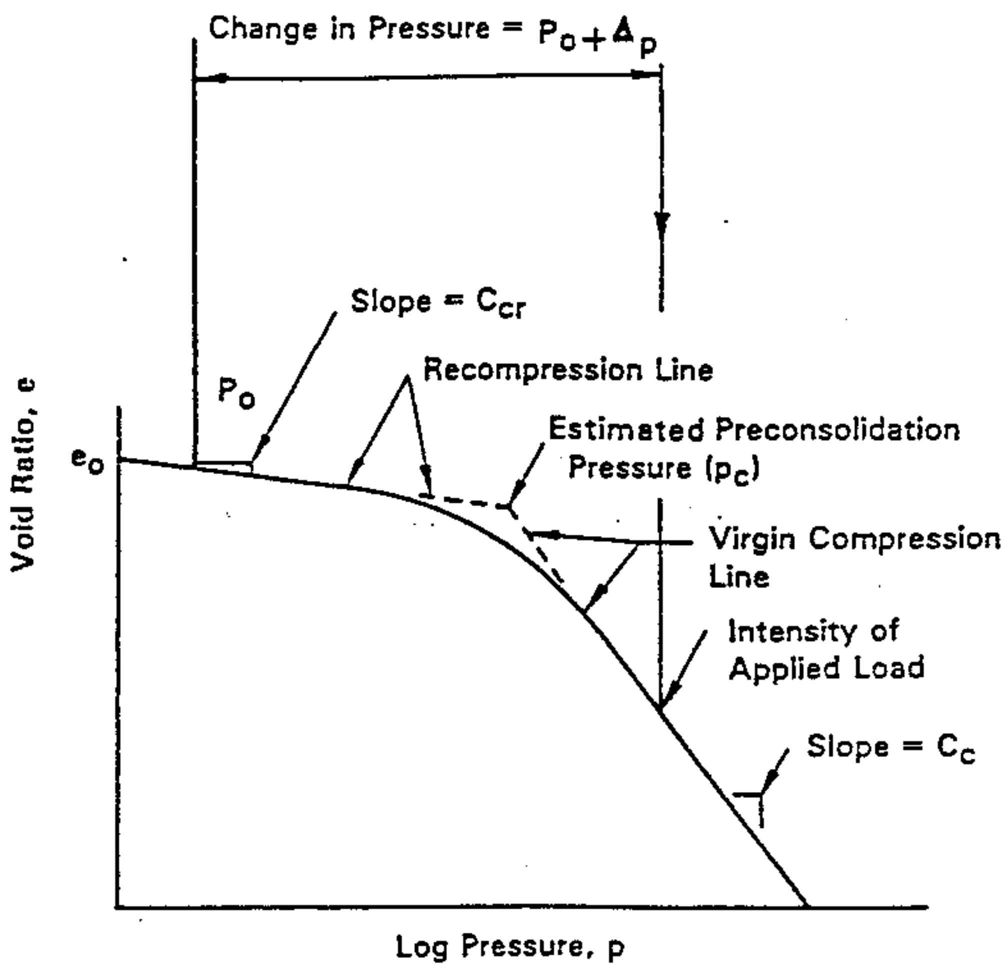


FIGURE 20. THE e-log p RELATIONSHIP (MODIFIED FROM CANADIAN GEOTECHNICAL SOCIETY, 1978)

Procedure for Estimating Pile Group Settlement in Cohesive Soil

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

a. Use the method shown in Figure 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.

b. Divide the cohesive soil layer determined in a. 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.

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

d. Determine the imposed pressure (p) at midpoint of each layer by using the method shown in Figure 19.

STEP 2: Determine Consolidation Test Parameters

a. Plot results of consolidation test (Figure 20)

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

STEP 3: Compute Settlements

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

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

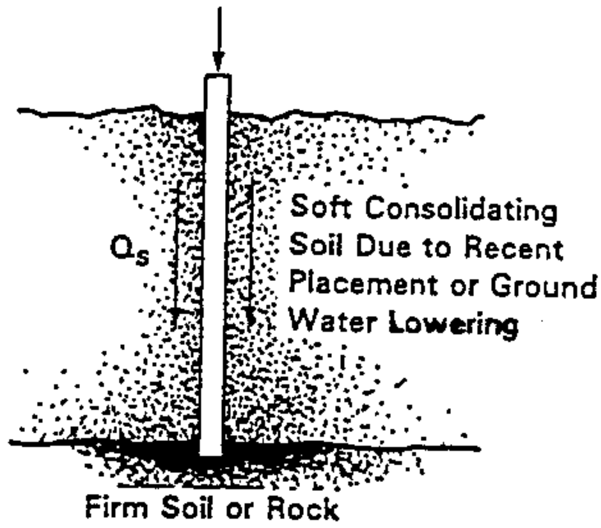
5. NEGATIVE SKIN FRICTION

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

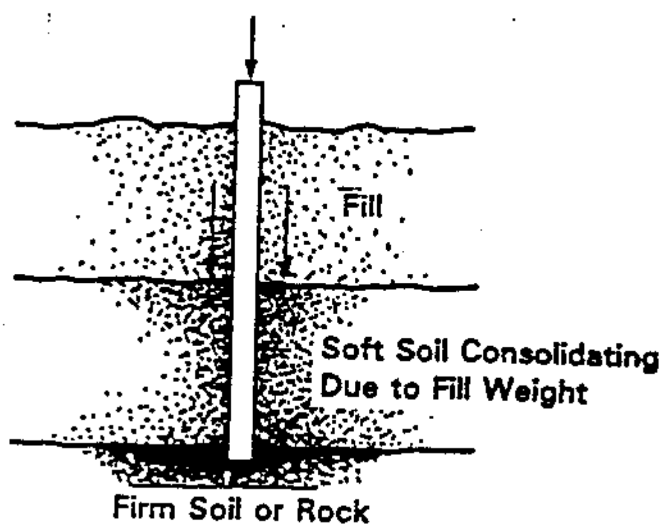
situations because of the additional bending forces imposed on the piles, which can result in the pile breaking.

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. <u>The amount of relative settlement between soil and pile that is necessary to fully mobilize negative skin friction is about 13 mm (1/2 inch).</u> 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 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.



(A)



(B)

FIGURE 21. NEGATIVE SKIN FRICTION SITUATIONS.

6. 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 22). If the horizontal movement is large, it may cause damage to structures. The unbalanced fill loads shown in Figure 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:

6.1 Determining Lateral Squeeze

Lateral squeeze or abutment tilting can occur if:

$(\gamma_{\text{Fill}} \times H_{\text{Fill}}) > (3 \times \text{undrained shear strength of soft soil})$

6.2 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

6.3 Solutions to Prevent Tilting

The following solutions are possible means of eliminating tilting:

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

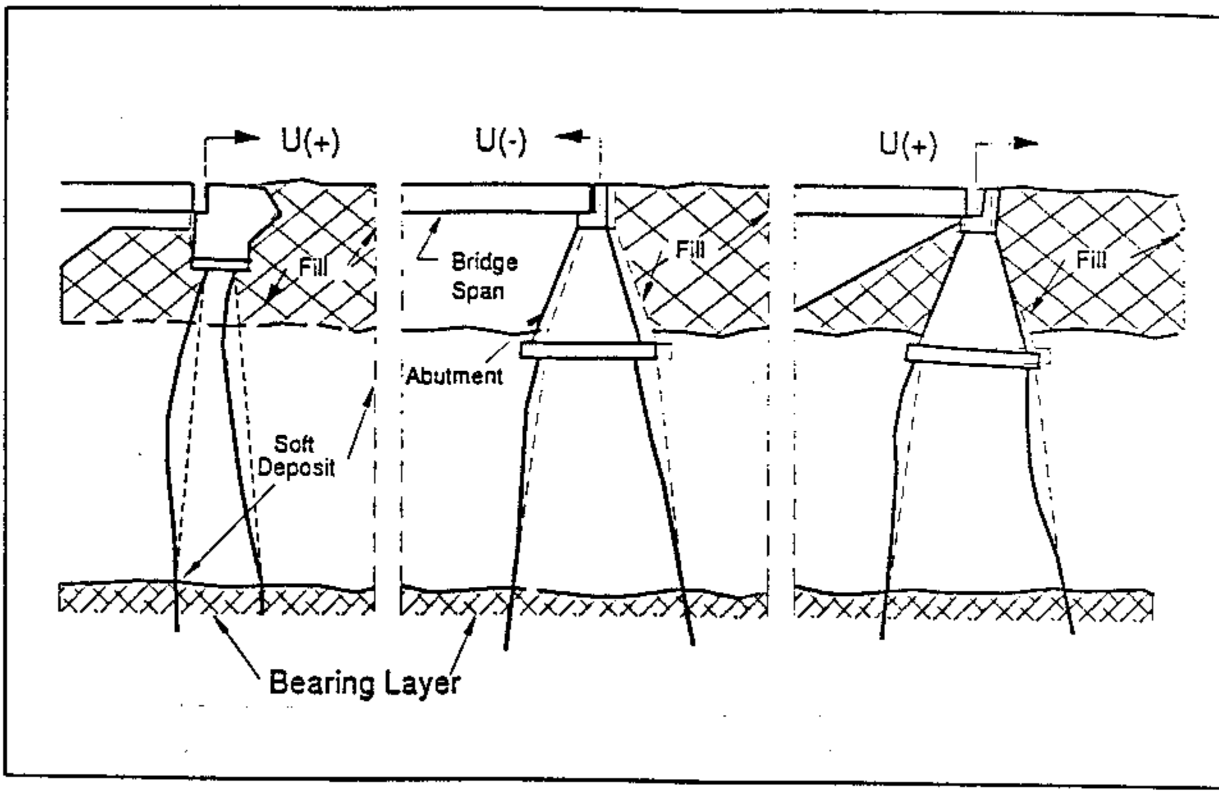


FIGURE 22. ABUTMENT TILTING DUE TO LATERAL SQUEEZE.

7. 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.

7.1 Parameters Effecting Lateral Load Capacity of Piles

Three types of parameters have significant effects on the lateral load capacity of piles. These three types of parameters are as follows:

7.1.1 Soil Parameters

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

b. 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.

7.1.2 Pile Parameters

a. Physical properties such as shape, material, and dimensions.

b. Pile head conditions such as free head or fixed head.

c. Method of placement such as jetting or driving.

d. Group action.

7.1.3 Load Parameters

a. Type of loading such as static (continuous) or dynamic (cyclic).

b. Eccentricity.

7.2 Design Methods

Three (3) basic design approaches are used in practice. They are lateral load tests, arbitrary values, and analytical methods.

7.2.1 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.

7.2.2 Arbitrary (Prescription) Values

Arbitrary values of lateral load capacity are empirical. They do not consider all the site parameters and may lead to overdesign or underdesign. 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 (see list of references) 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 without special analysis or design features."

SOURCE	PILE TYPE	DEFLECTION	ALLOWABLE LATERAL LOADS		
		in (mm)	lbs		(kg)
NYSDOT	TIMBER CONCRETE STEEL	--- --- ---	10,000 15,000 20.000		(4500) (6800) (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 TIMBER CONCRETE CONCRETE	1/4 (6.5) 1/2 (12.5) 1/4 (6.5) 1/2 (12.5)	9,000 14,000 12,000 17,000		(4100) (6300) (5400) (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

TABLE 2. Prescription Values For Allowable Lateral Loads On Vertical Piles (After New York, State Department of Transportation, 1977).

7.2.3 Analytical Methods

The analytical methods are based on theory and empirical data and permit the inclusion of various site parameters. Two (2) available approaches are (1) Brom's method and (2) 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. Reese's more sophisticated methods include analysis by computer (COM-624 Program) and a non-dimensional method which does not require computer use. 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, 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 (4) or five (5) 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)
n_1 and n_2 = empirical coefficients taken from Table 3.

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

TABLE 3. Values of Coefficients n_1 and n_2 For Cohesive Soils

or the coefficient of horizontal subgrade reaction K_h within the critical depth from a cohesionless soil.

Cohesionless Soils: Choose K_h from the Table 4 (Terzaghi)

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

TABLE 4. Values of K_h For Cohesionless Soils.

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 in 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 (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. $\beta = 4 \quad K_h D / 4 E I$ for cohesive soil, or
b. $n = 5 \quad K_h / E I$ for cohesionless soil.

STEP 6: The Dimensionless Length Factor:

Determine the dimensionless length factor:

a. βL for cohesive soil, or

b. $n L$ 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) $nL > 4.0$ (long pile)

(2) $nL < 2.0$ (short pile)

(3) $2.0 < nL < 4.0$ (intermediate pile)

STEP 8: Other Soil Parameters:

Determine other soil parameters:

a. Rankine passive pressure coefficient for cohesionless soil, $K_p = \tan^2 (45 + \phi/2)$ where ϕ = angle of internal friction.

b. Average effective soil unit weight over embedded length of pile γ (kg/m^3) or (pcf).

c. Cohesion, C_u = one-half unconfined compressive strength, ($q_u/2$) (kN/m^2) or (psi).

STEP 9: Ultimate (Failure) Load for a Single Pile:

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

a. Short Free or Fixed-Headed Pile in Cohesive Soil:

Using L/D (and e/D for the free-headed case), enter Figure 17, select the corresponding value of $P_u / C_u D^2$, and solve for P_u (kg) or (lb.).

b. 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 26, select the corresponding value of $P_u / C_u D^2$, and solve for P_u (kg) or (lb.).

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

Using L/D (and e/L for the free-headed case), enter Figure 19, select the corresponding value of $Pu/KpD_3\gamma$ and solve for Pu (kg) or (lb.).

d. 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 20, select the corresponding value of $P_u/K_pD^3\gamma$ and solve for P_u (kg) or (lb.).

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

Calculate P_u for both a short pile (step 9c) and a long pile (step 9d) and use the smaller value.

STEP 10: Maximum Allowable Working Load for a Single Pile

Calculate the maximum allowable working load for a single pile P_m from the ultimate load P_u determined in Step 9 (this is shown in Figure 29):

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

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

Calculate the working load for a single pile P_a corresponding to a given design deflection at the ground surface y, or the deflection corresponding to a given design load. If Pa and y are not given,

substitute the value of P_m (kg) or (lb) from Step 10 for Pa in the following cases and solve for Y_m (m) or (in.):

a. Free or Fixed-Headed Pile in Cohesive Soil:

Using L (and e/L for the free-headed case), enter Figure 23, select the corresponding value of YK_hDL/P_a, and solve for P_a(kg) or (lb) or y (m) or (in.).

b. Free or Fixed-Headed Pile in Cohesionless Soil:

Using nL (and e/L for the free-headed case), enter Figure 24, select the corresponding value of $y\,(E\,I)^{3/5}\,K_h^{2/5}/P_a\,L$, solve for P_a (kg) or (lb) or y (m) or (in.).

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

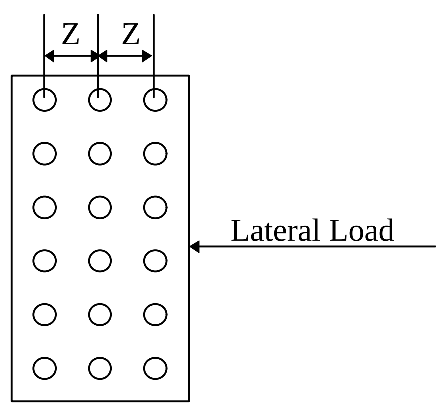
If $P_a < P_m$, use P_a and Y.

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

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

a. Group effects as determined by pile spacing Z in the direction of load:

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



b. Method of installation -- for driven piles use no reduction, and for jetted piles use 0.75 of the value from Step 13a.

STEP 14: The total lateral load capacity of the pile group equals the adjusted allowable load per pile from Step 13b 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.

Special Note

Inspection of Figures 27 and 28 for cohesionless soils indicates that the ultimate 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 3) and thus the deflection will be greater than for the dry state.

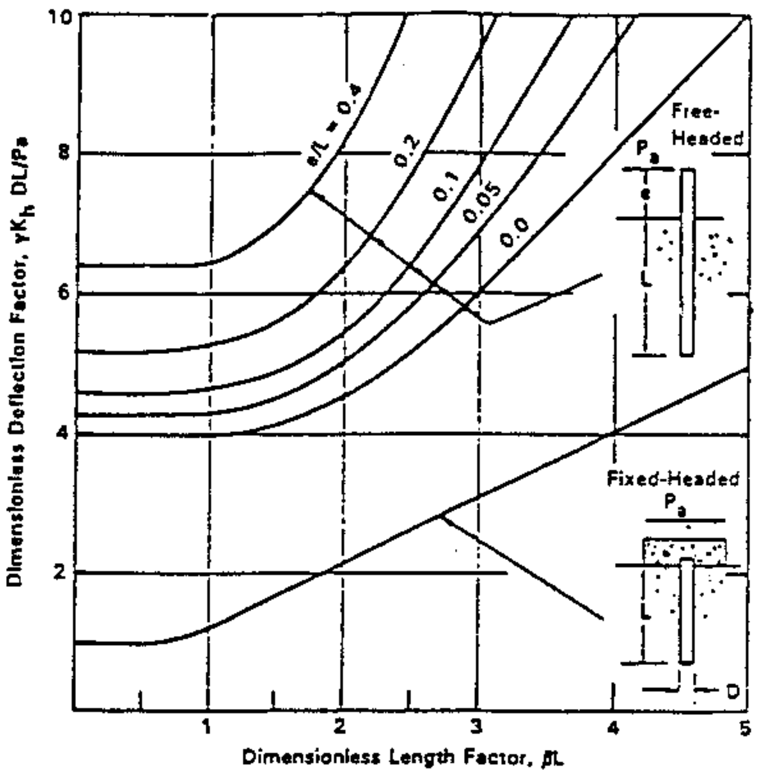


FIGURE 23. LATERAL DEFLECTIONS, AT GROUND SURFACE, OF PILES IN COHESIVE SOILS

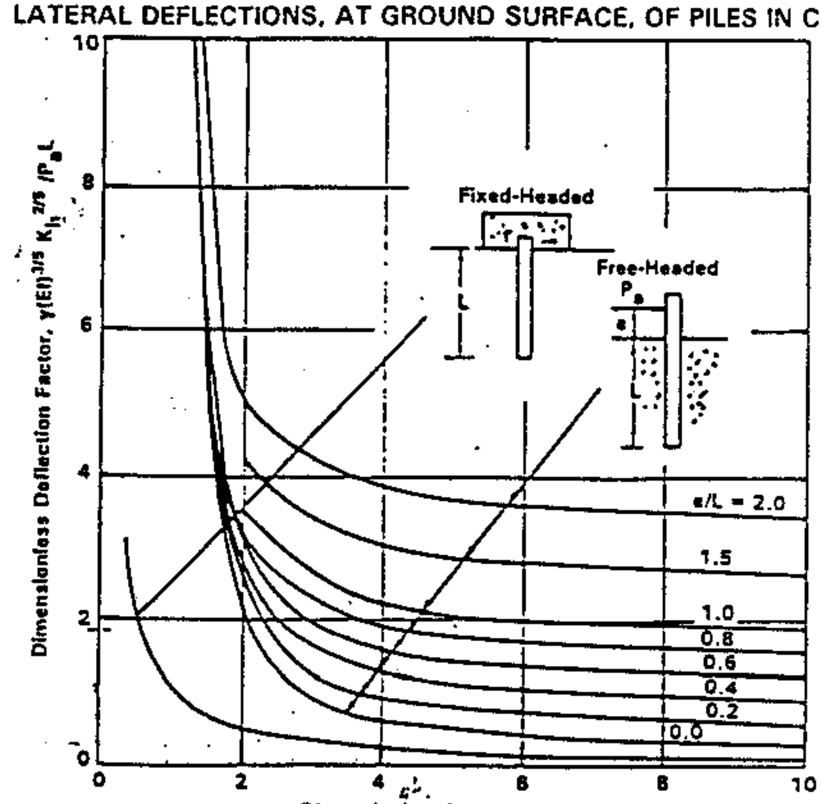


FIGURE 24. LATERAL DEFLECTIONS, AT GROUND SURFACE, OF PILES IN COHESIONLESS SOILS

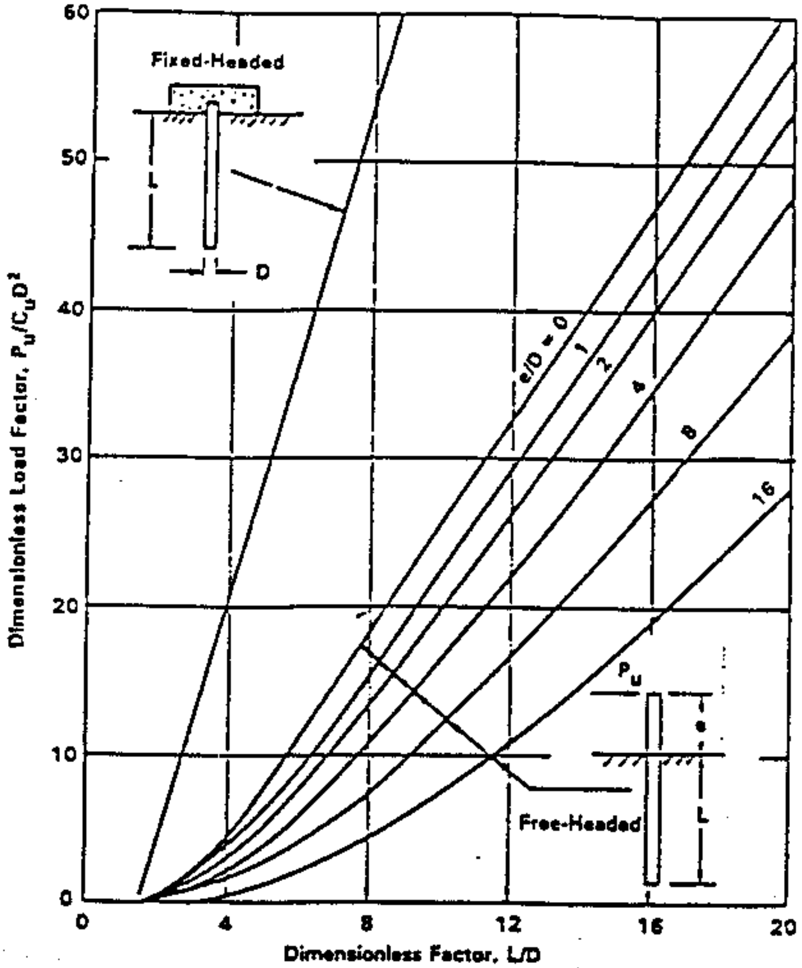


FIGURE 25. ULTIMATE LATERAL LOAD CAPACITY OF SHORT PILES IN COHESIVE SOILS

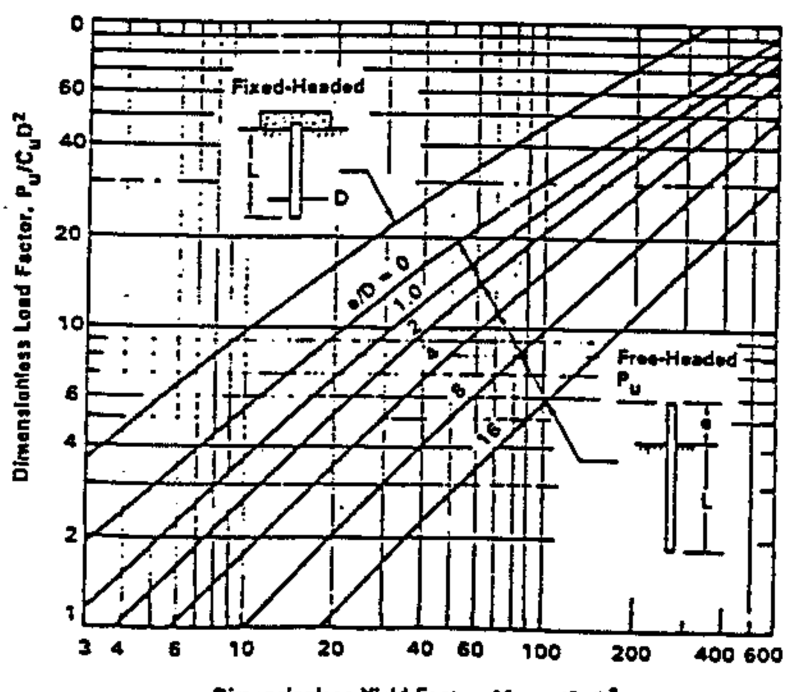


FIGURE 26. ULTIMATE LATERAL LOAD CAPACITY OF LONG PILES IN COHESIVE SOILS

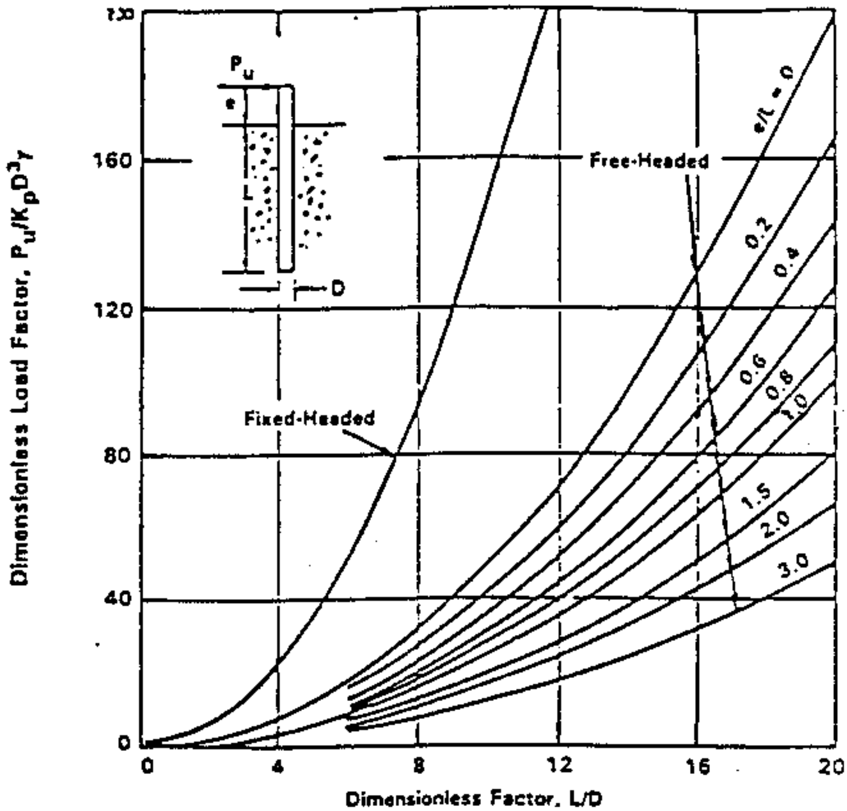


FIGURE 27. ULTIMATE LATERAL LOAD CAPACITY OF SHORT PILES IN COHESIONLESS SOILS

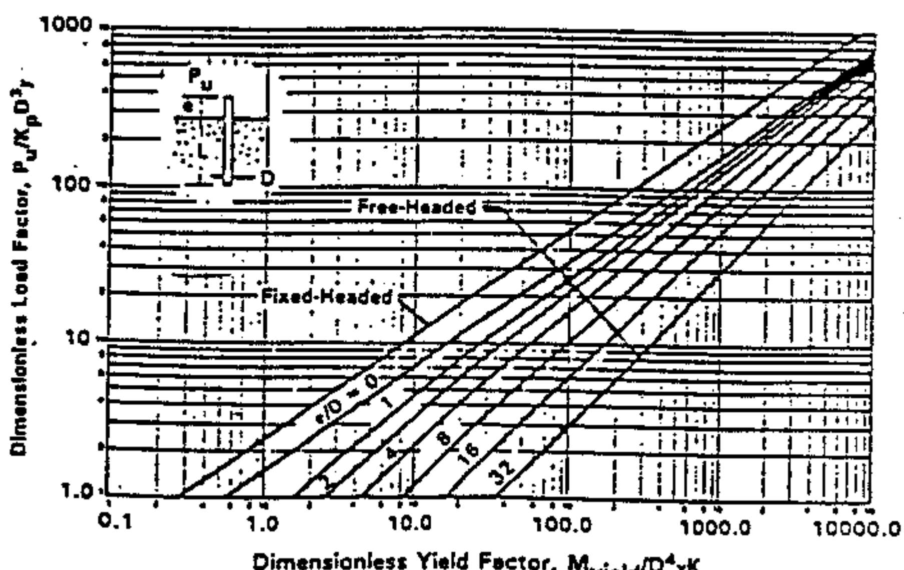


FIGURE 28. ULTIMATE LATERAL LOAD CAPACITY OF LONG PILES IN COHESIONLESS SOILS

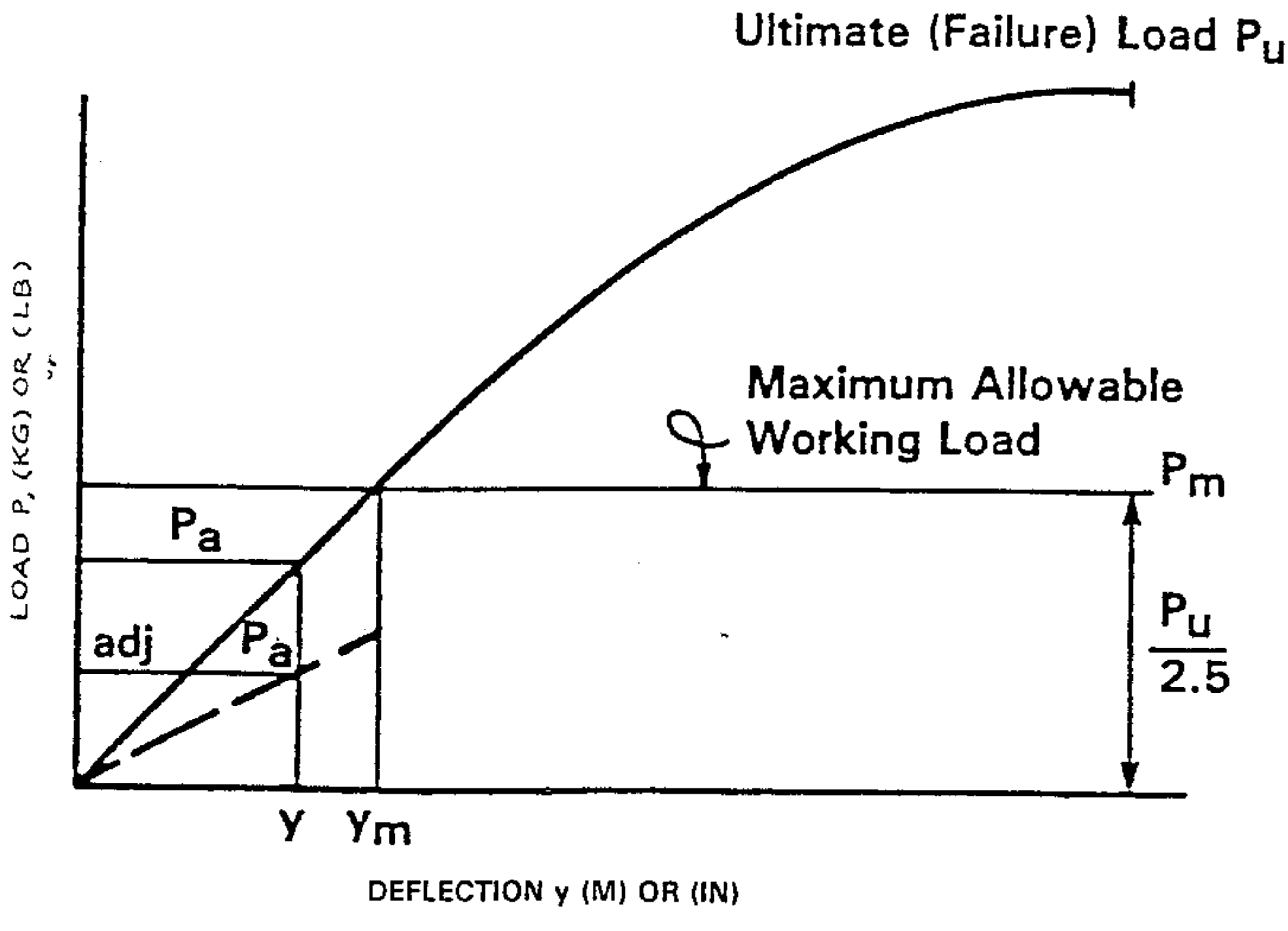


FIGURE 29. RELATIONSHIP OF LOAD AND DEFLECTION

7.2.4 Reese's COMP642P Method

The interaction of a pile-soil system subjected to lateral load has long been recognized as a complex function of nonlinear response characteristics. The most widely used nonlinear analysis method is the p-y method, where p is the soil resistance per unit pile length and y is the lateral soil or pile deflection. This method, illustrated in Figure 30, models the soil resistance to lateral load as a series of nonlinear springs.

Reese (1984, 1986) has presented procedures for describing the soil response surrounding a laterally loaded pile for various soil conditions by using a family of p-y curves. The procedures for constructing these curves are based on experiments using full-sized, instrumented piles and theories of the behavior of soil under stress.

The soil modulus E_s, is defined as follows:

$$E_s = - p / y$$

The negative sign indicates that the soil resistance opposes pile deflection. The soil modulus, E_s, is the modulus of the p-y curve and is not constant except over a small range of deflections. Typical p-y curves are shown in Figure 30. Ductile p-y curves, such as curve A, are typical of the response of soft clays under static loading and sands. Brittle p-y curves, such as curve B, can be found in some stiff clays under dynamic loading conditions.

The COM624 program solves the nonlinear differential equations representing the behavior of the pile-soil system to lateral (shear and moment) loading conditions in a finite difference formulation using Reese's p-y method of analysis. The strongly nonlinear reaction of the surrounding soil to pile-soil deflection is represented by the p-y curve prescribed to act on each discrete element of the embedded pile. For each set of applied boundary (static) loads the program performs an iterative solution which satisfies static equilibrium and achieves an acceptable compatibility between force and deflection (p and y) in every element.

The shape and discrete parameters defining each individual p-y curve may be input by the analyst, but are most often generated by the program. Layered soil systems are characterized by conventional geotechnical data including soil type, shear strength, density, depth, and stiffness parameters, and whether the loading conditions are monotonic or cyclic in nature.

In version 2.0 of the COM624P, the influence of applied loads (axial, lateral and moment) at each element can be modeled with flexural rigidity varying as a function of applied moment. In this manner, progressive flexural damage such as cracking in a reinforced concrete pile can be treated more rigorously. The COM624P program code includes a subroutine (PMEIX) which calculates the value of flexural rigidity at each element under the boundary conditions and resultant pile-soil interaction conditions.

COM624P problem data is input through a series of menu-driven screens. In most cases help screens are available. Detailed information concerning the software can be found in the FHWA publication FHWA-SA-91-048, COM624P - Laterally Loaded Pile Program for the Microcomputer, Version 2.0, by Wang and Reese (1993). Part I provides a User's Guide, Part II presents the theoretical background on which the program is based, and Part III deals with System Maintenance. The appendices include useful guidelines for integrating COM624P analyses into the overall design process for laterally loaded deep foundations, and a comprehensive case study example implementing the design guidelines.

The COM624P computer printout file summarizes the input information and the analysis results. The input data summarized includes the pile geometry and properties, and soil strength data. Output information includes the generated p-y curves at various depths below the pile head and the computed pile deflections, bending moments, stresses and soil moduli as functions of depth below the pile head. The information allows an analysis of the pile's structual capacity.

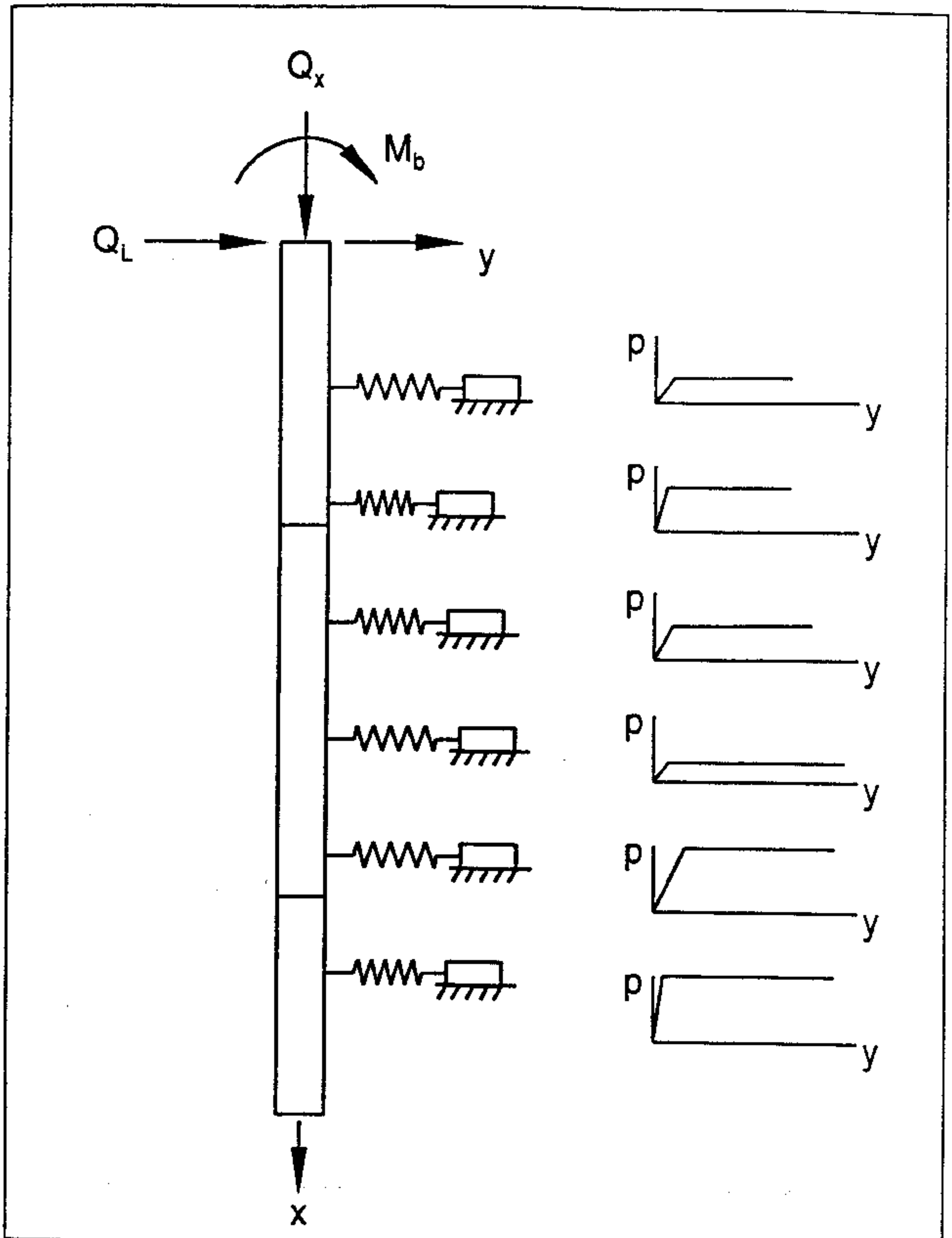


FIGURE 30 COM624P PILE-SOIL MODEL

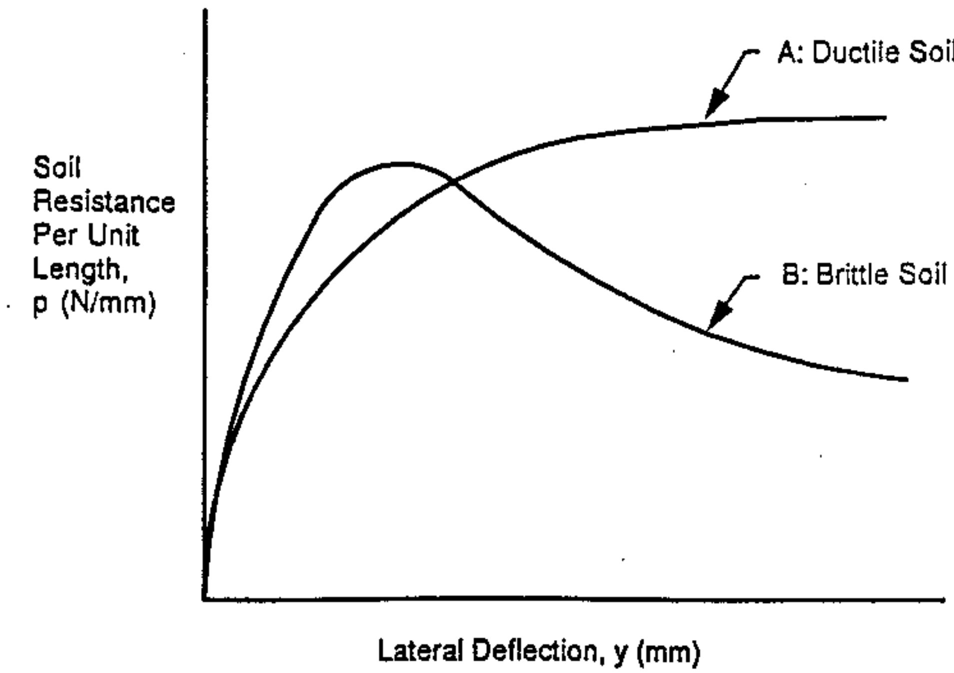


FIGURE 31 TYPICAL p-y CURVES FOR DUCTILE AND BRITTLE SOIL (AFTER CODUTO 1994)

The factor most influencing the shape of the p-y curve is the soil properties. However, the p-y curves also depend upon depth, soil stress-strain relationships, pile width, water table location, and loading conditions (static or cyclic). Procedures for constructing p-y curves for various soil and water table conditions as well as static or cyclic loading conditions are provided in the COM624P program documentation by Wang and Reese (1993) FHWA-SA-91-048.

Procedures for p-y curve development cover the following soil and water table conditions:

1. Soft clays below the water table.

2. Stiff clays below the water table.

3. Stiff clays above the water table.

4. Sands above or below the water table.

Internally generated (or input) values of flexural rigidity for cracked or damaged pile sections are also output. Graphical presentations versus depth include the computed deflection, slope, moment, and shear in the pile, and soil reaction forces similar to those illustrated in Figure 32.

The COM624P analyses characterize the behavior of a single pile under lateral loading conditions. A detailed view is obtained of the load transfer and structural response mechanisms to design conditions. Considerable care is required in extrapolating the results to the behavior of pile groups (pile-soil-pile interaction, etc.), and accounting for the effects of different construction processes such as predrilling or jetting.

In any lateral analysis case, the analyst should verify that the intent of the modeling assumptions, all elastic behavior for example, is borne out in the analysis results. When a lateral load test is performed, the measured load-deflection results versus depth should be plotted and compared with the COM624P predicted behavior so that an evaluation of the validity of the p-y curves used for design can be made, such as that presented in Figure 33.

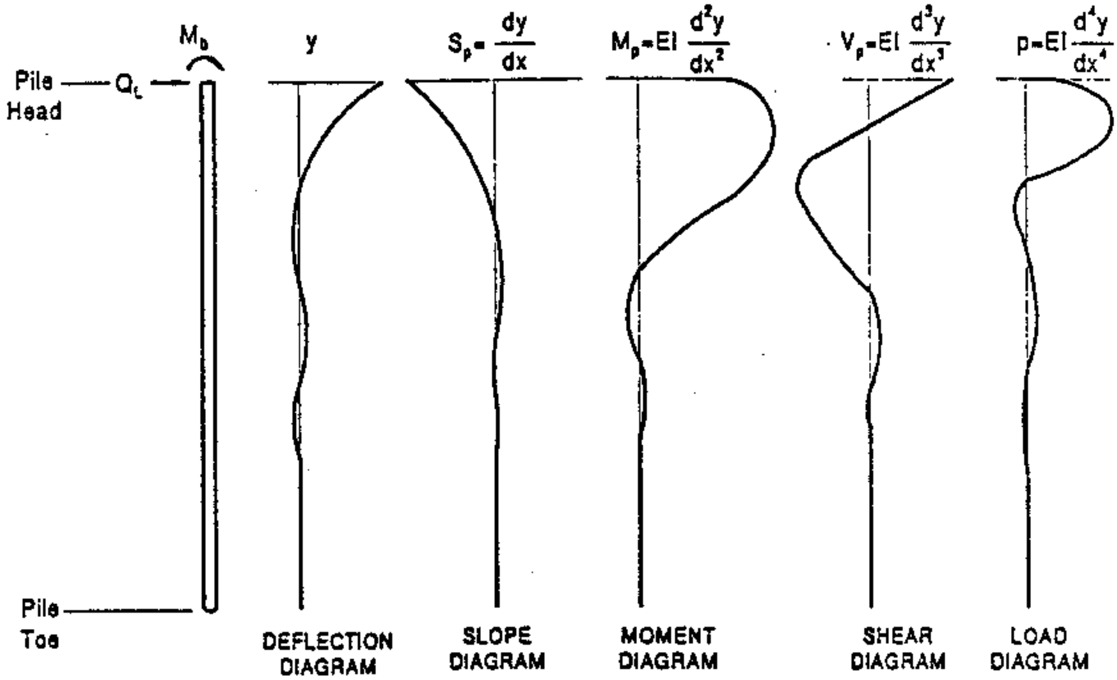


FIGURE 32 GRAPHICAL PRESENTATION OF COM624P RESULTS (AFTER REESE, 1986)

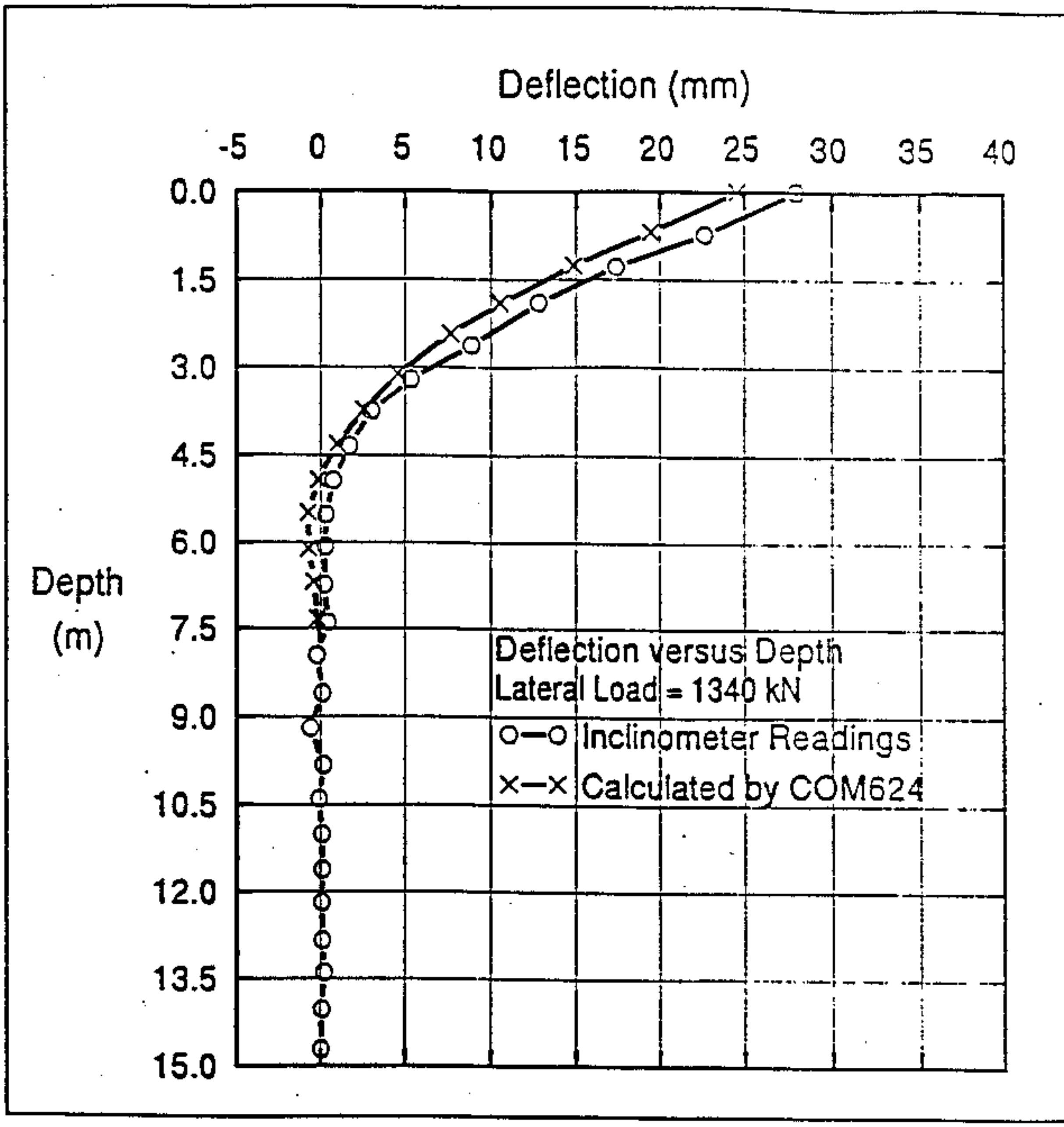


FIGURE 33 COMPARISON OF MEASURED AND COM624P PREDICTED LOAD-DEFLECTION BEHAVIOR VERSUS DEPTH (AFTER KYFOR et al. 1992)

STEP BY STEP PROCEDURE FOR USING THE COM624P PROGRAM

STEP 1 Determine basic pile input parameters for trial pile.

a. Pile length (m)

b. Modulus of elasticity, E (kPa).

c. Distance from pile head to ground surface (m).

d. Number of increments for pile model (300 maximum).

e. Slope of the ground surface, if any, (degrees).

STEP 2 Divide pile into segments with uniform cross sectional properties. For each segment, provide:

a. X-coordinate at top of segment.

b. Pile diameter (m).

c. Moment of inertia,I, (m^4).

d. Area of pile (m^2).

STEP 3 Delineate the soil profile into layers over the maximum anticipated penetration depth of the trial pile. Soil profile delineation should include:

a. Location of the ground water table.

b. Top and bottom depth of each soil layer from the ground surface (m).

c. Soil layer characterization as cohesive or cohesionless.

STEP 4 Determine the required soil input parameters for each layer.

a. Soil effective unit weights, γ (kN/m^3)

b. Soil strength parameters.

1. - For cohesive layers:
   - cohesion, c_u (kPa), and

ε_{50}, the measured strain at 1/2 maximum principle stress from triaxial tests or an assumed value from Table 5.

2. - For cohesionless layers:

- ϕ angle from laboratory, in-situ data, or SPT N values.

c. Slope of soil modulus (kN/m^3) measured from laboratory or in-situ test data or assumed value from Table 6.

CLAY CONSISTENCY	AVERAGE UNDRAINED SHEAR STRENGTH, c_u (kpa)	ϵ_{50}
Soft Clay	12 - 24	0.02
Medium Clay	24 - 48	0.01
Stiff Clay	48 - 96	0.007
Very Stiff Clay	96 - 192	0.005
Hard Clay	192 - 383	0.004

TABLE 5: REPRESENTATIVE VALUES OF ε_{50} FOR CLAYS

SOIL TYPE	AVERAGE UNDRAINED SHEAR STRENGTH, c_u (kpa)	SOIL CONDITION	k-Static Loading (kN/m^3)	k-Cyclic Loading (kN/m^3)
Soft Clay	12 - 24	- - -	8,140	- - -
Medium Clay	24 - 48	- - -	27,150	- - -
Stiff Clay	48 - 96	- - -	136,000	54,300
Very Stiff Clay	96 - 192	- - -	271,000	108,500
Hard Clay	192 - 383	- - -	543,000	217,000
Loose Sand	- - -	Submerged	5,430	5,430
Loose Sand	- - -	Above Water Table	6,790	6,790
Med. Dense Sand	- - -	Submerged	16,300	16,300
Med. Dense Sand	- - -	Above Water Table	24,430	24,430
Dense Sand	- - -	Submerged	33,900	33,900
Dense Sand	- - -	Above Water Table	61,000	61,000

TABLE 6 REPRESENTATIVE k VALUES FOR CLAYS AND SANDS

STEP 5 Develop p-y curves for selected depths. Decide if program or user input p-y curves will be used.

a. Program p-y curves can be input at user selected depths. Curves are assigned to soil layers using a criteria number.

b. User p-y curves require input of deflection (m) and soil resistance (kN/m) coordinates for each p-y curve at user selected depths.

STEP 6 Determine the critical loading combinations and boundary conditions to be analyzed.

a. For each critical loading combinations, determine the axial loads, lateral loads, and bending moments to be analyzed. Load information should be supplied by the structural engineer.

b. Determine if lateral load is distributed.

c. Determine if loading is static or cyclic.

d. Determine pile head restraint: free, fixed or partial fixed.

STEP 7 Determine pile structural acceptability by finding the ultimate lateral load that produces a plastic hinge (ultimate bending moment).

a. In this step the lateral, axial and bending moments used in the analysis should be ultimate values.

b. For concrete piles, the value of I for a cracked section can be determined directly for each loading step by using the subroutine PMEIX, through identification of the properties and configuration of the steel reinforcement. Alternatively, variations in E and I can be entered as a function of depth along the pile.

STEP 8 Determine pile acceptability based on deflection under service loads.

a. Use design loading conditions and not ultimate values for lateral and axial loads and bending moments.

b. Compare COM624P predicted movement with performance criteria.

STEP 9 Optimize required pile section and pile penetration depth for lateral loading conditions to meet performance criteria as necessary.

8. 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.

Bridge Memorandum No. 213 and Figure 30 summarize the INDOT Bridge Designs recommendations for seismic activities. Text from the original memo No. 213 (Dec. 21, 1983) and its most recent update (Dec. 1, 1992) is being reproduced here for reference. All consultants should obtain and review the recommended publications mentioned in the memorandum.

BRIDGE MEMORANDUM #213

December 21, 1983

Effective January 1, 1984, all structures which have not yet received Design Approval shall be designed in accordance with the requirements of the AASHTO Guide Specifications for Seismic Design of Highway Bridges. However, the attached map of earthquake acceleration coefficients shall be used instead of the map shown in the Guide Specifications.

Earthquake design criteria were first adopted by AASHTO in the 1958 Interim Bridge Specifications, but it was not clear if these applied to Indiana. Revised criteria clearly affecting Indiana were adopted in the 1975 Interim AASHTO Bridge Specifications. The new Guide Specifications were adopted in the 1983 Interim AASHTO Specification as an alternate to the 1975 criteria. Because the Guide Specifications represent the latest consensus and advances in the state of the art, the INDOT has elected to make the Guide Specifications mandatory for all structure designs instead of an optional alternate. The new Guide Specifications have in fact already been used for several designs on critical facilities in the southwest portion of the State. This memo now extends their use to the entire State.

Even though the Guide Specifications have been adopted, they have not yet been published by AASHTO, and the publication date and price are not yet known. However, the specifications were developed by the Applied Technology Council for the Federal Highway Administration under the title "Seismic Design Guidelines for Highway Bridges". Copies of this document are available for inspection from Mr. Steven Hull and Mr. Jack White in the Bridge Design Office. Consultants may order their own copies from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161. The price if $19.00 and the publication number is PB 82-181611

The Guidelines closely parallel a similar document, "Tentative Provisions for the Development of Seismic Regulations for Buildings", also developed by the Applied Technology Council. For those who are interested, a copy is available for inspection at the State Library as catalog number C 13.10:510. The contour map of acceleration coefficients in the AASHTO Guidelines was

developed from a county-by-county map of coefficients in the Building Guidelines. Because interpolation between contours implies an accuracy not present in the original map, the INDOT has elected to use the county-by-county values given in the Building Guidelines document.

W. B. Abbott
Engineer of Bridge Design

SJH:mjc

FIGURE 34. Earthquake Design Acceleration Coefficients.

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