SOILS AND FOUNDATIONS
Lesson 09

Chapter 9 – Deep Foundations

Lesson Plan

- Topic 1 (Section 9.0 to 9.8)
  - Driven piles
  - Static capacity
- Topic 2 (Section 9.9)
  - Driven Piles - Construction Monitoring and QA
- Topic 3 (Section 9.9.10)
  - Driven Piles – Load Tests
- Topic 4 (Section 9.10)
  - Drilled shafts
  - Static capacity
  - Construction

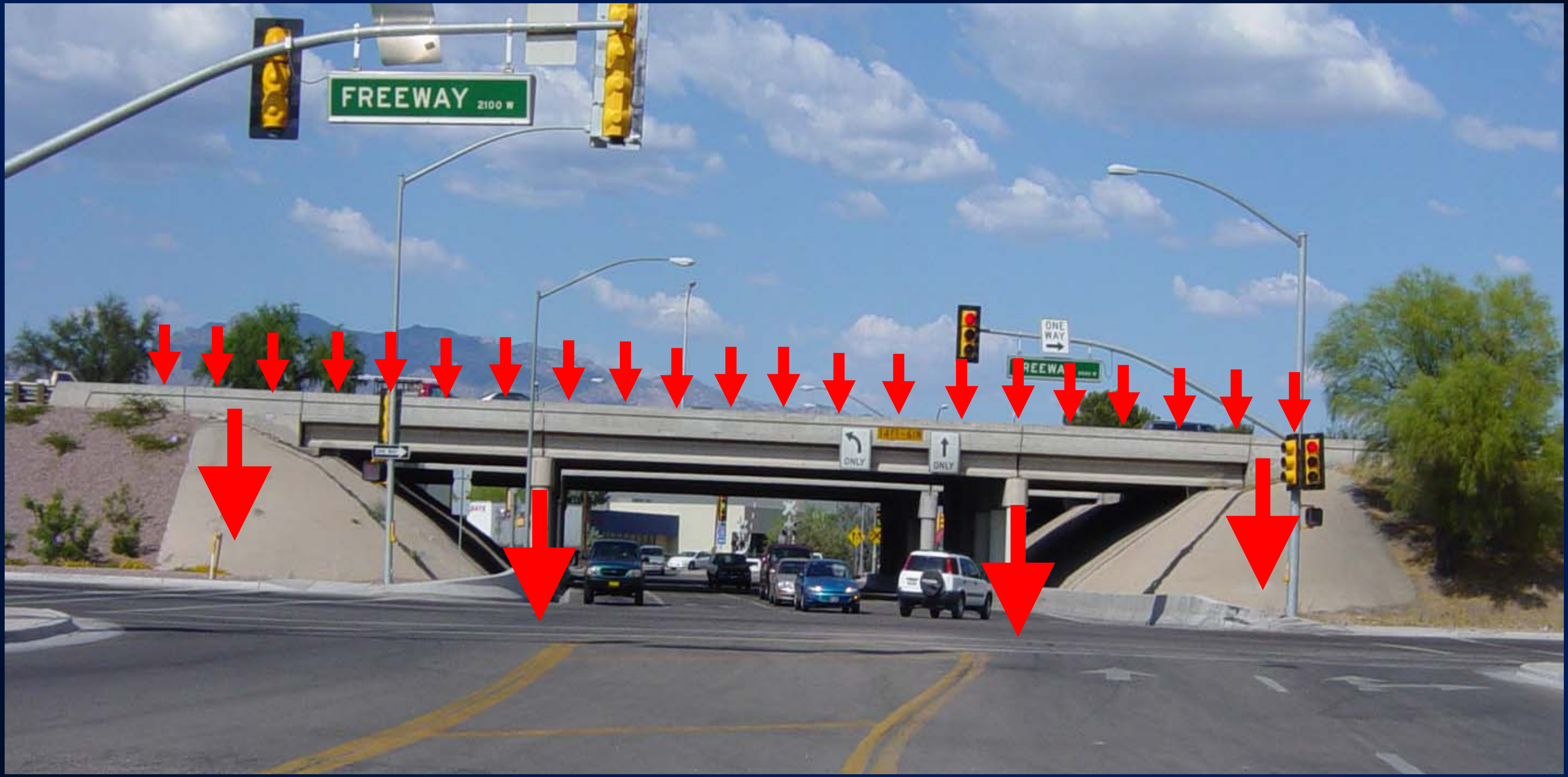
Deep Foundations

Lesson 09 - Topic 1
Driven Piles and Static Capacity
Section 9.0 to 9.8

Learning Outcomes

- ***At the end of this session, the participant will be able to:***
  - ***Describe types of driven piles and applications for use***
  - ***Compute static capacity for driven piles in granular and cohesive soils***
  - ***Identify 2 of the design steps in pile groups***
  - ***Discuss negative skin friction***

Stresses Imposed by Structures



- ***Deep foundations may be used at pier and abutment locations***

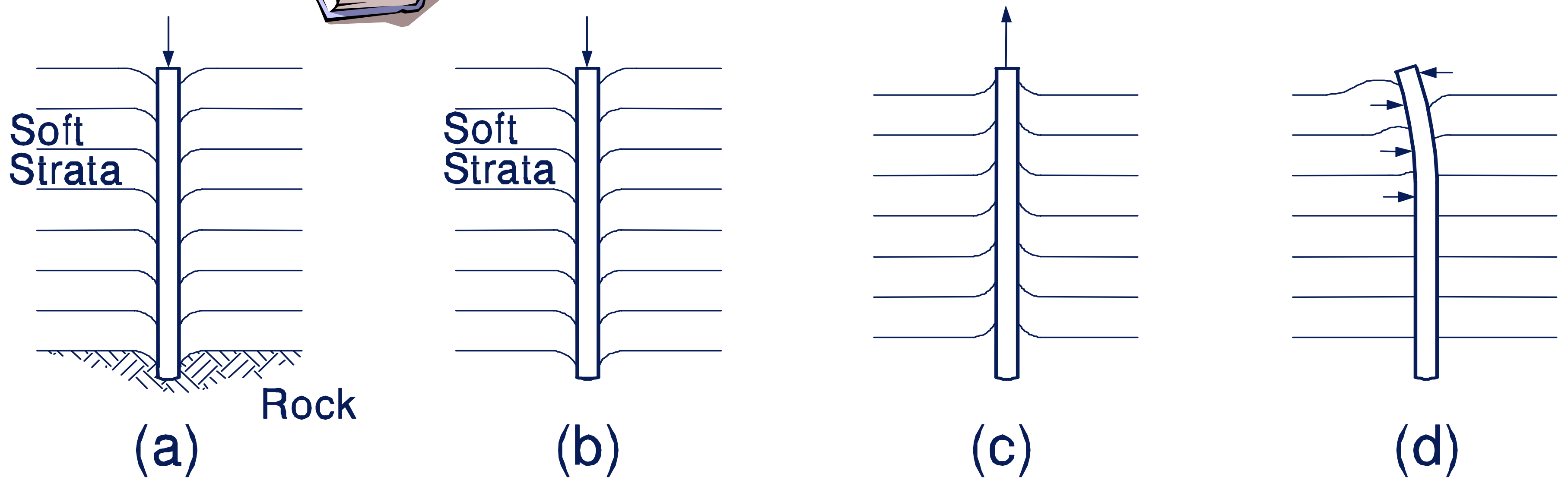
Establishment of need for deep foundations

Structural Foundation Topics

- Shallow Foundations (Spread Footings)
  - Bearing Capacity
  - Settlement
- Deep Foundations
  - Load Capacity
  - Settlement
  - Negative Skin Friction

Situations Where a Deep Foundation is Needed

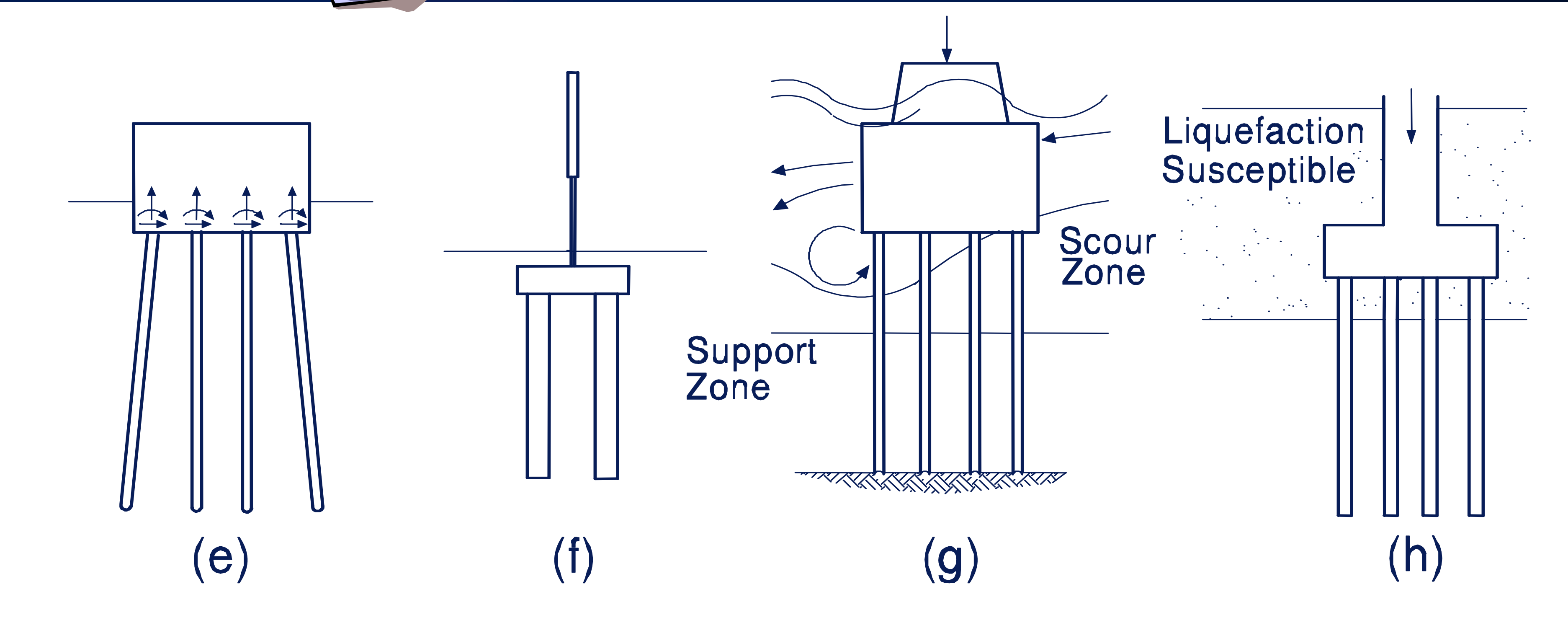
Figure 9-1



- *The foundation designer must define at what depth suitable soil layers begin in the soil profile*

Situations Where a Deep Foundation is Needed

Figure 9-1



Situations Where a Deep Foundation is Needed

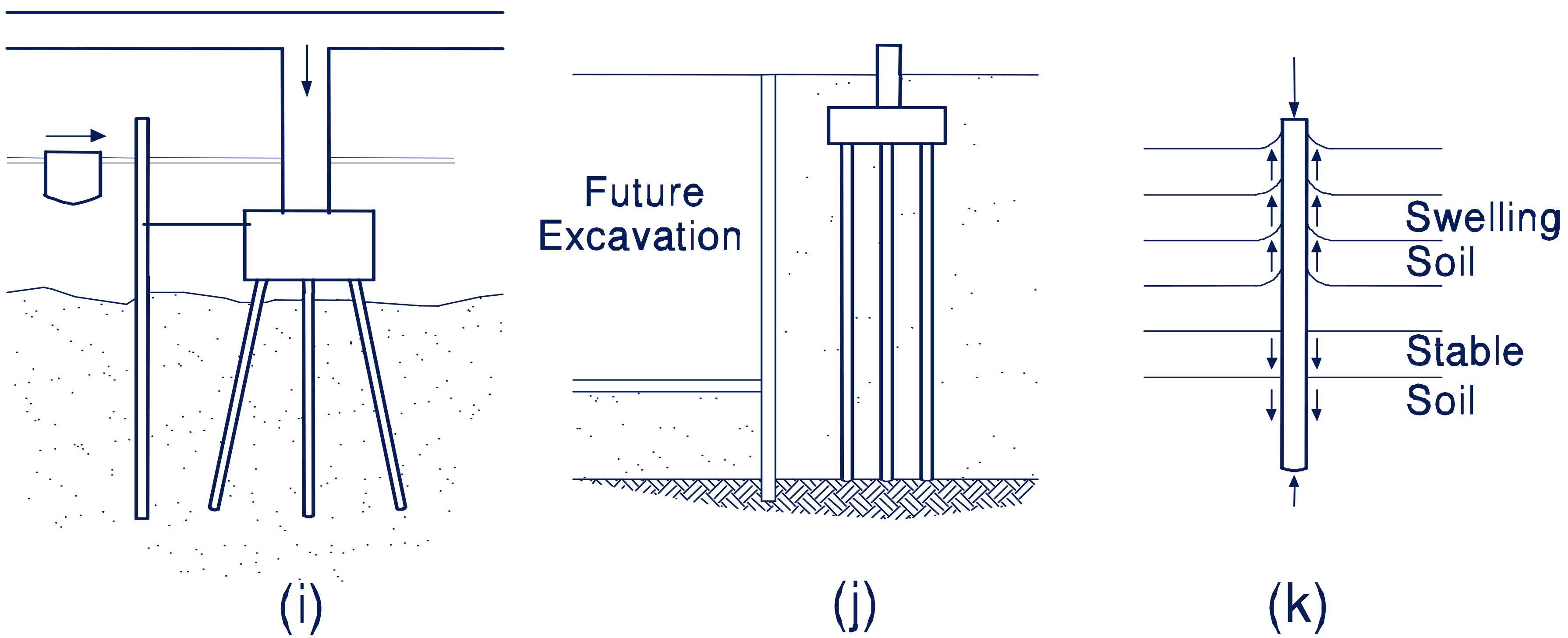
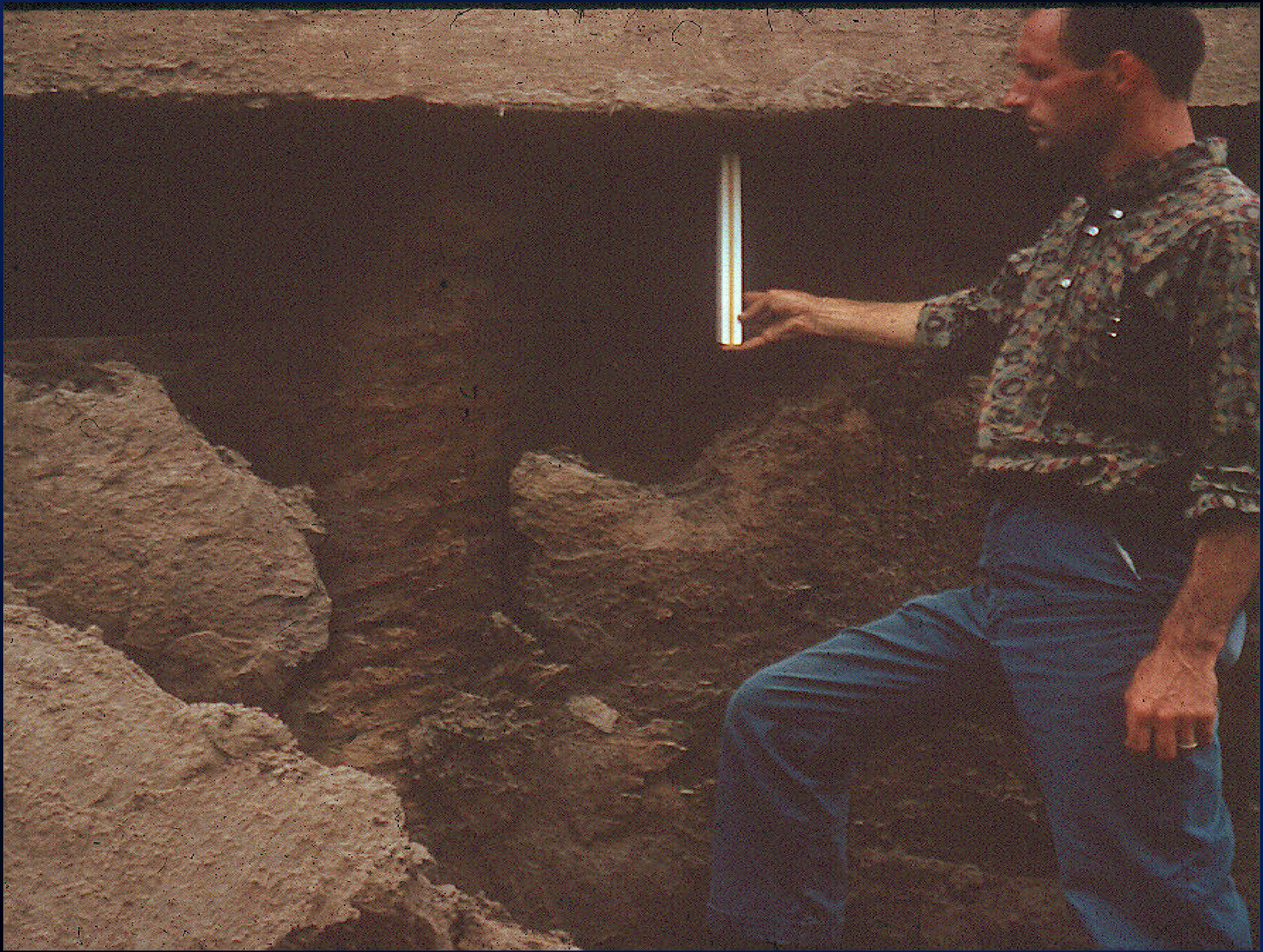


Figure 9-1


F
EDG 857



Deep Foundation Classification System

- *Figure 9-2* 

Design and Construction Geotechnical Terminology

- ***Specific terminology for deep foundations***
  - ***Static pile capacity***
  - ***Ultimate pile capacity***
  - ***Driving capacity***
  - ***Restrike capacity***
  - ***Shaft resistance in piles***
  - ***Side resistance in drilled shafts***
  - ***Toe resistance for piles***
  - ***Tip or base resistance for shafts***
  - ***And more…….***

Structural Terminology

- Allowable load
- Design load
  - Equal to or less than allowable load
- Ultimate (Nominal) load
- Table 9-10 for maximum structural design stress and maximum structural driving stress

Types of Piling

Typical Pile Information

- *Table 9-1*

Type of Pile	Typical Axial Design Loads	Typical Lengths
Timber	20-110 kips (100 – 500 kN)	15-120 ft (5-37 m)*
Precast / Prestressed Reinforced Concrete	90-225 kips (400-1000 kN) for reinforced 90-1000 kips (400-4500 kN) for prestressed	30-50 ft (10-15m) for reinforced 50-130 ft (15-40m) for prestressed
Steel H	130-560 kips (600-2500 kN)	15-130 ft (5-40 m)
Steel Pipe (without concrete core)	180-560 kips (800-2500 kN)	15-130 ft (5-40 m)
Steel Pipe (with concrete core)	560-3400 kips (2500-15000 kN)	15-130 ft (5-40 m)
* 15-75 ft (5-23 m) for Southern Pine; 15-120 ft (5-37 m) for Douglas Fir		

Effect of Subsurface and Hydraulic Conditions on Piles

- *Table 9-2*

Typical Problem	Recommendations
Boulders overlying bearing stratum	Use heavy nondisplacement pile with a reinforced tip or manu-factured point and include con-tingent predrilling item in contract.
Loose cohesionless soil	Use tapered pile to develop maximum skin friction.
Negative skin friction	Use smooth steel pile to minimize drag adhesion, and avoid battered piles. Minimize the magnitude of drag force when possible.
Deep soft clay	Use rough concrete pile to increase adhesion and rate of pore water dissipation.
Artesian Pressure	Do not use mandrel driven thin-wall shells as generated hydrostatic pressure may cause shell collapse; pile heave common to closed-end pipe.
Scour	Do not use tapered piles unless large part of taper extends well below scour depth. Design permanent pile capacity to mobi-lize soil resistance below scour depth.
Coarse Gravel Deposits	Use precast concrete piles where hard driving expected in coarse soils. DO NOT use H-piles or open end pipes as nondisplacement piles will penetrate at low blow count and cause unnecessary overruns.

Pile Shape Effects

- ***Table 9-3***

Shape Characteristics	**Pile Types**	**Placement Effects**
Displacement	Steel Pipe (Closed end), Precast Concrete	•Increase lateral ground stress •Densify cohesionless soils, remolds and weakens cohesive soils temporarily •Set-up time may be 6 months in clays for pile groups
Nondisplacement	Steel H, Steel Pipe (Open end)	•Minimal disturbance to soil •Not suited for friction piles in coarse granular soils. Piles often have low driving resistances in these deposits making field capacity verification difficult thereby often resulting in excessive pile lengths.
Tapered	Timber, Monotube, Tapertube, Thin-wall shell	•Increased densification of soils with less disturbance, high capacity for short length in granular soils

Other issues

- Noise and vibrations during installation
- Remote areas may restrict driving equipment size
- Local availability of certain materials
- Waterborne operations may dictate some handling limitations (e.g., shorter pile sections)
- Steep terrain may make use of certain pile equipment costly or impossible

Cost Evaluation of Alternate Deep Foundation Types

- Often several deep foundation types meet project requirements
- Final choice must be made on cost analysis
- In cost analysis include ALL costs related to a given pile type
  - Uncertainties in execution, time delays, cost of load testing, cost of pile caps, noise and vibrations, etc

Cost Evaluation of Alternate Pile Types

- Three major categories of cost for driven piles
  - Pile support cost
  - Pile cap support cost
  - Construction control method support cost

- For most piles, the pile cost is usually linear with depth based on unit price
  - May not be true for very long concrete or long, large section steel piles
    - Special handling, splicing, etc.

Cost Evaluation of Alternate Pile Types

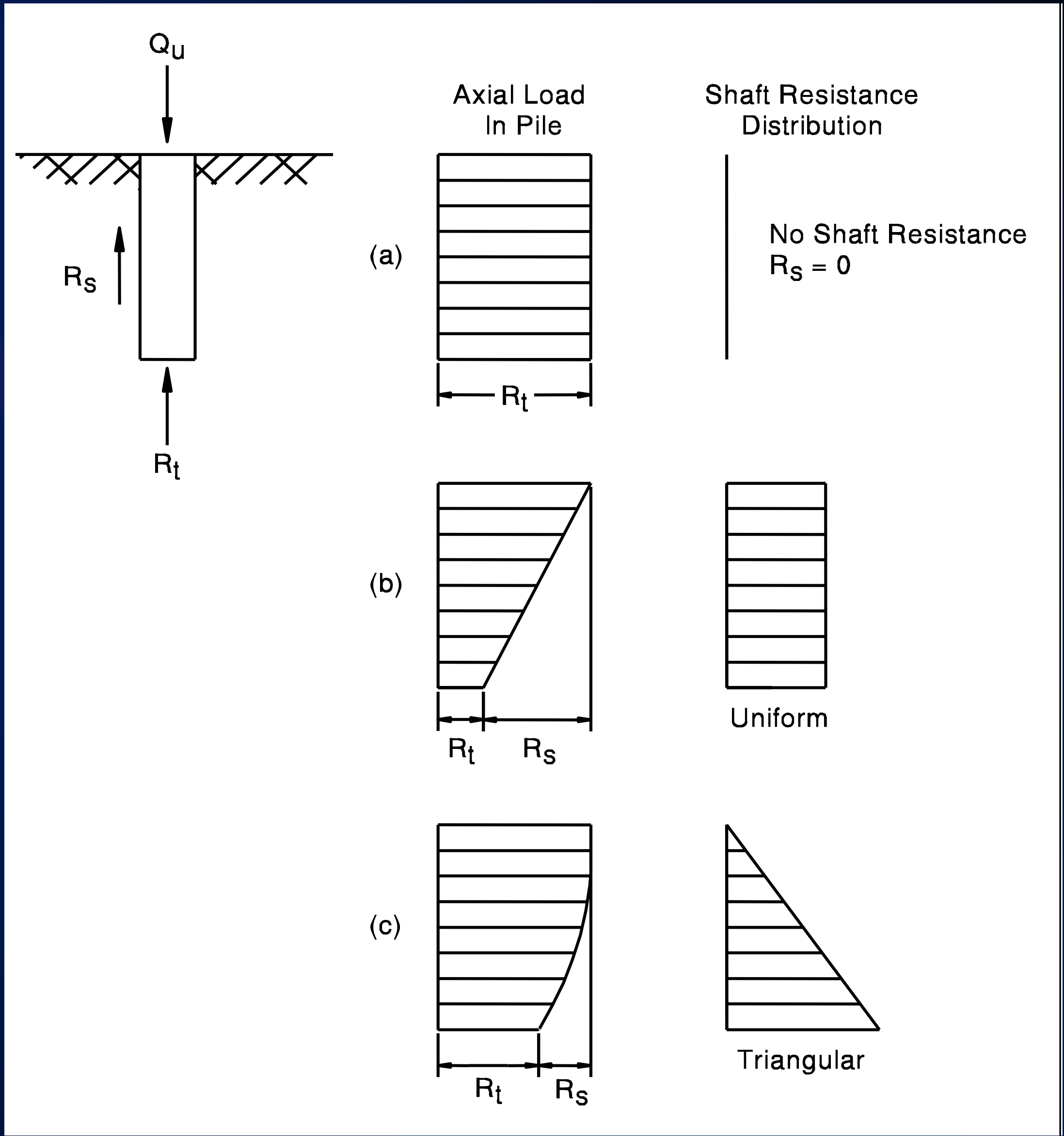
- Express costs in terms of $/ton capacity for each alternative as discussed in Chapter 8

- For cost savings recommendations, see Table 9-4 

Computation of Pile Capacity

- *Ultimate pile capacity, Q_u*
- *Shaft resistance, R_s*
- *Toe resistance, R_t*

- *$Q_u = R_s + R_t$*



Computation of Pile Capacity

- Shaft resistance, $R_s = f_s A_s$

  f_s is unit shaft resistance
  A_s is shaft surface area

- Toe resistance, $R_t = q_t A_t$

  q_t is unit toe resistance
  A_t is pile toe area

- $Q_u = R_s + R_t = f_s A_s + q_t A_t$

Allowable Geotechnical Pile Load

- *The allowable geotechnical pile load, Q_a is defined as follows in terms of Q_u and factor of safety, FS*

$$Q_a = \frac{Q_u}{\text{Factor of Safety}}$$

Factor of Safety

- FS depends on the following:
  - Level of confidence in input parameters
  - Variability of soil and rock
  - Method of static analysis
  - Proposed pile installation method
  - Level of construction monitoring

- The FS used in static analysis should be based upon the construction control method specified

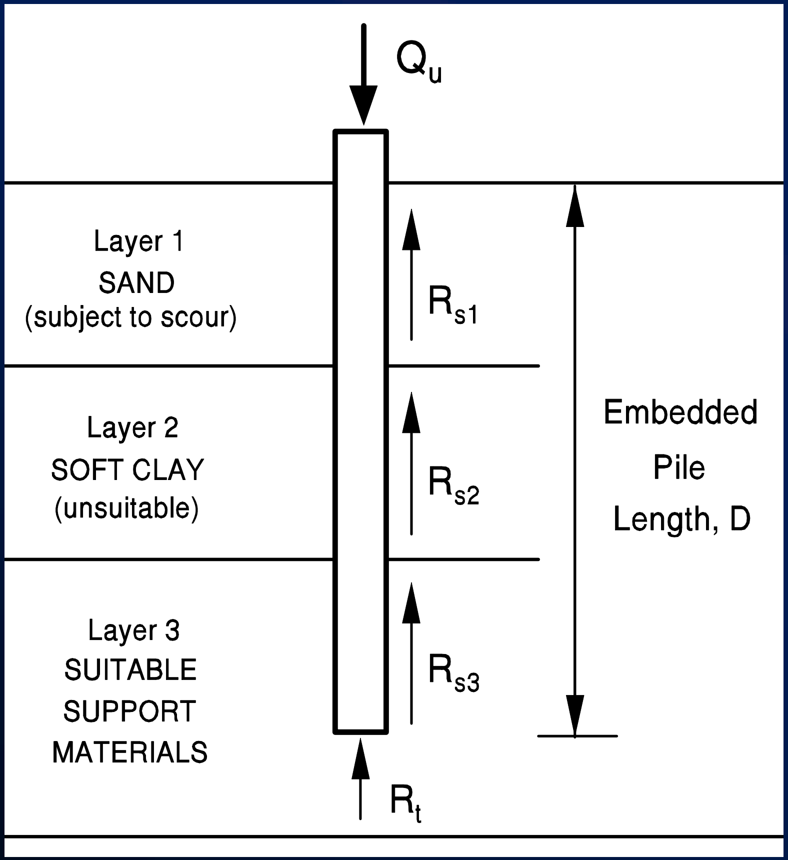
Factor of Safety (Table 9-5)

Construction Control Method	Factor of Safety
Static load test (ASTM D-1143) with wave equation analysis	2.00
Dynamic testing (ASTM D-4945) with wave equation analysis	2.25
Indicator piles with wave equation analysis	2.50
Wave equation analysis	2.75
Gates dynamic formula	3.50


P&H


LE 725-9023

FS as a function of Soil Resistance



$$Q_u = R_{s1} + R_{s2} + R_{s3} + R_t$$

Assume static load test

For design

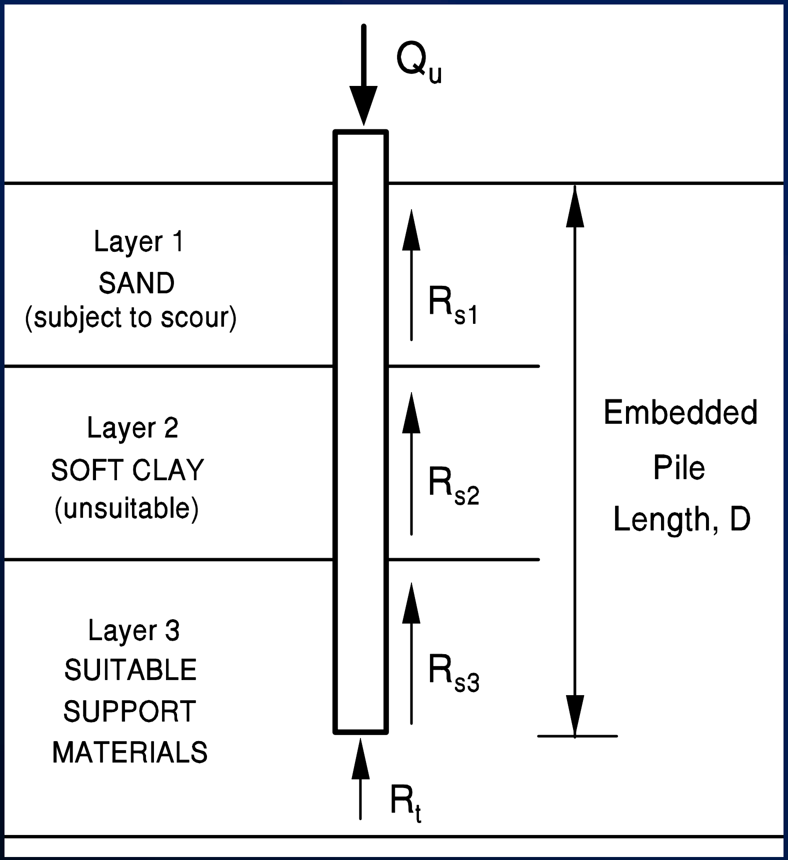
$$Q_a = (R_{s3} + R_t) / (FS=2)$$

$$Q_a = (Q_u - R_{s1} - R_{s2}) / (FS=2)$$

For plans and specs

$$Q_u = R_{s1} + R_{s2} + (Q_a)(FS=2)$$

Soil Driving Resistance (SRD)



In SRD, FS is not used

$$SRD = R_{s1} + R_{s2} + R_{s3} + R_t$$

<u>Soil Setup and Relaxation are considered in SRD</u>

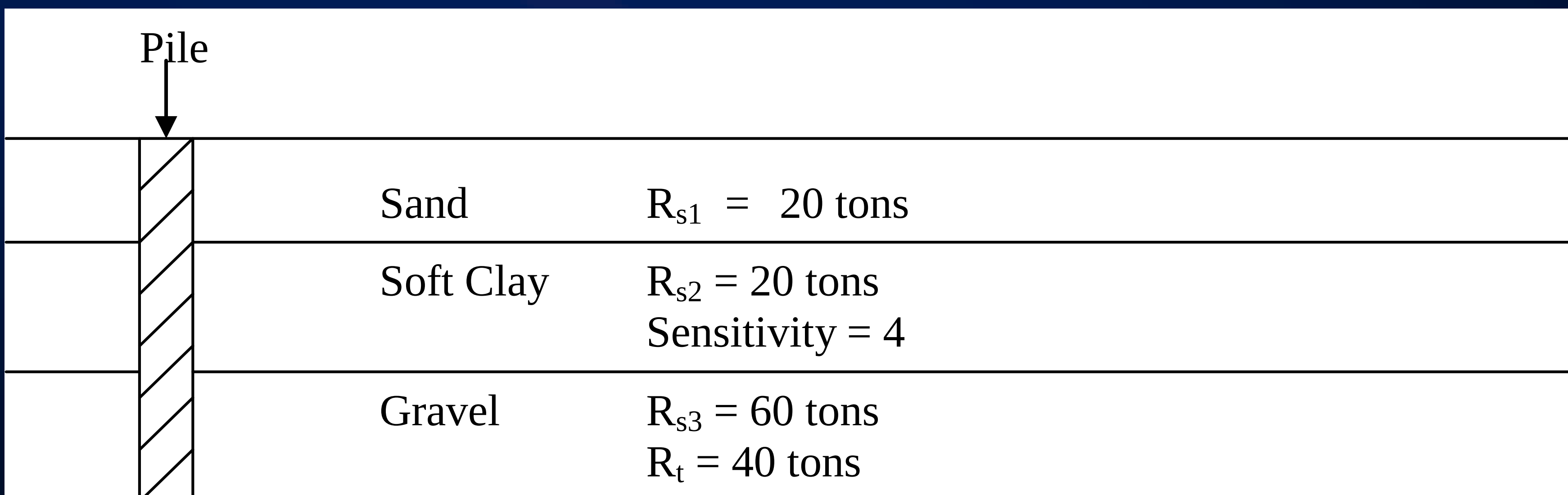
Assume soft clay layer has sensitivity of 2

$$SRD = R_{s1} + R_{s2}/2 + R_{s3} + R_t$$

<u>SRD should also include resistance to penetrate hard or dense layers</u>

Example 9-1

- ***Compute ultimate capacity and driving capacity***



Design of Single Piles

- *Cohesionless Soils*
- *Cohesive Soils*
- *Rocks*

Static Pile Capacity

Kerisel

Hansen

Caquot

Vijayvergiya & Focht

Tomlinson

Thurman

Broms

Kishida

Berezantsev et al

Meyerhof

Vesic

Nordlund

Nottingham & Schmertmann

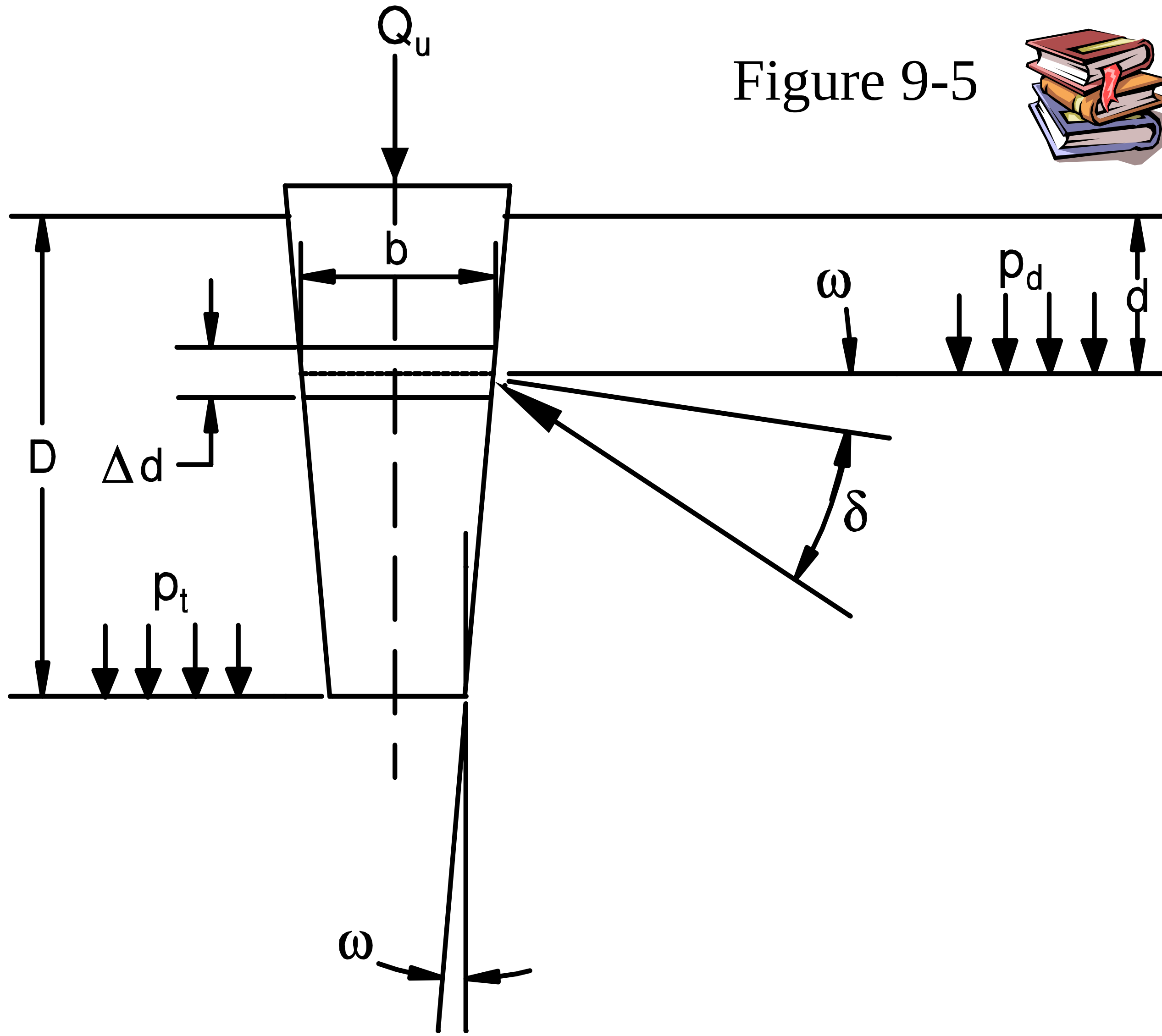
Seymour-Jones

Burland

Single Piles in Cohesionless Soils

Nordlund's Method

Figure 9-5



$$Q_u = \sum_{d=0}^{d=D} K_\delta C_F p_d \frac{\sin(\delta + \omega)}{\cos \omega} C_d \Delta d + \alpha_t N'_q A_t p_t$$

Nordlund Method

For a pile of uniform cross section (ω=0) and embedded length D, driven in soil layers of the same effective unit weight and friction angle, the Nordlund equation becomes:

$$Q_u = \underbrace{(K_\delta\, C_F\, p_d\, \sin\delta\, C_d\, D)}_{R_S} + \underbrace{(\alpha_t\, N'_q\, A_t\, p_t)}_{R_T}$$

Nordlund Shaft Resistance

$$R_s = K_\delta \, C_F \, p_d \, \sin\delta \, C_d \, D$$

K_δ = coefficient of lateral earth pressure — Figures 9.7 - 9.10

C_F = correction factor for K_δ when $\delta \neq \phi$ — Figure 9.11

p_d = effective overburden pressure at center of layer

δ = friction angle between pile and soil — Figure 9.9

C_d = pile perimeter

D = embedded pile length

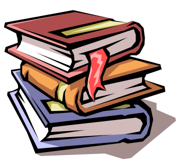
Nordlund Toe Resistance

Lesser of

$$R_t = \alpha_t N'_q p_t A_t$$

$$R_t = q_L A_t$$

α_t = dimensionless factor Figure 9.12a



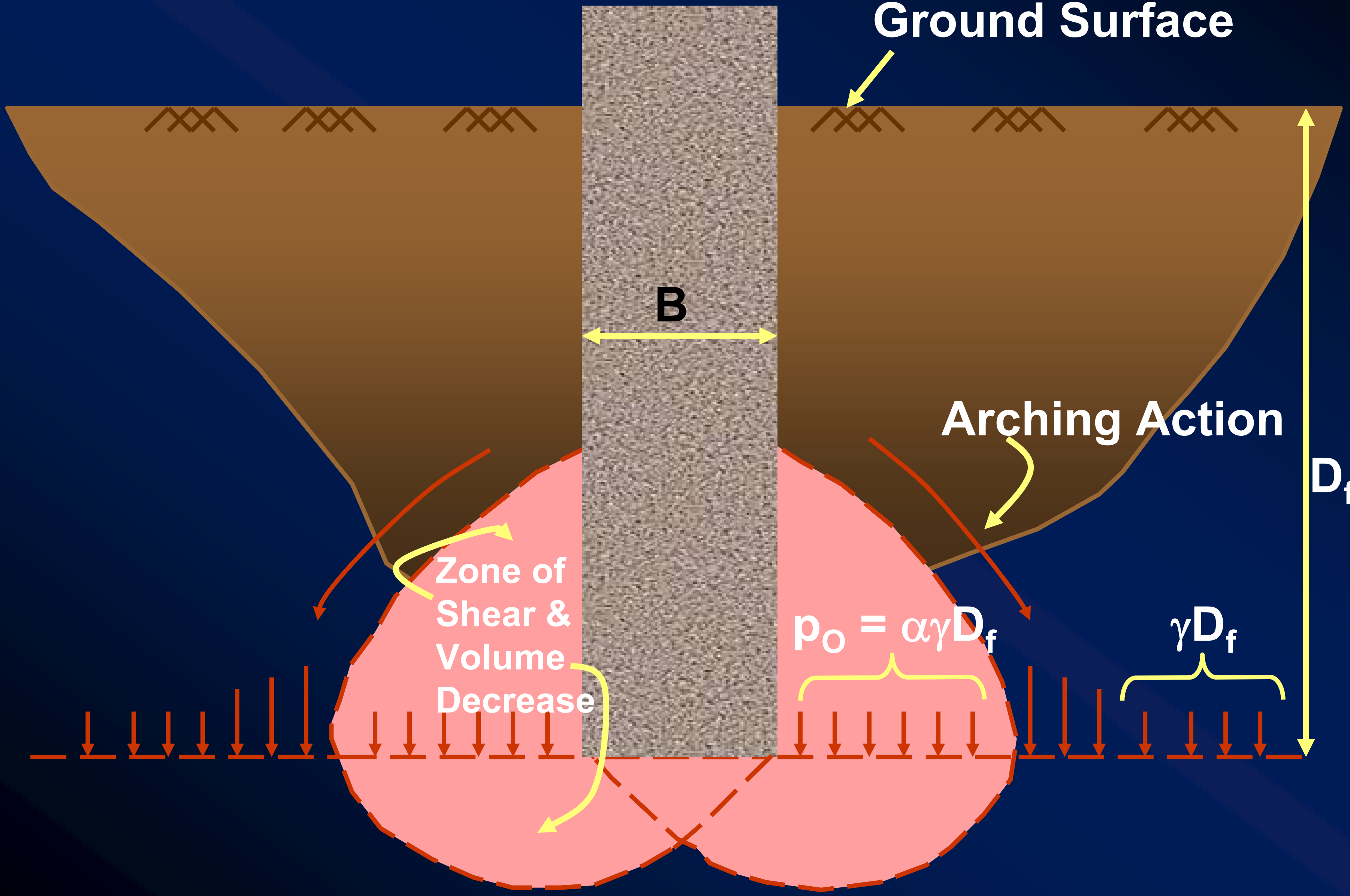
N'_q = bearing capacity factor Figure 9.12b

A_t = pile toe area

p_t = effective overburden pressure at pile toe $\leq$ 3 ksf

q_L = limiting unit toe resistance Figure 9.17

Arching at Pile Tip



Nordlund Method

$$Q_u = R_S + R_T$$

and

$$Q_a = Q_U / FS$$

FS based on construction control method as in Table 9-5

Nordlund Method Procedure

Steps 1 through 6 are for computing shaft resistance and steps 7 through 9 are for computing the pile toe resistance

STEP 1 Delineate the soil profile into layers and determine the ϕ angle for each layer

a. Construct p_o diagram using procedure described in Chapter 2.

b. Correct SPT field N values for overburden pressure using Figure 3-23 from Chapter 3 and obtain corrected $N1_{60}$ values. Delineate soil profile into layers based on corrected $N1_{60}$ values.

c. Determine ϕ angle for each layer from laboratory tests or in-situ data.

d. In the absence of laboratory or in-situ test data, determine the average corrected $N1_{60}$ value, N', for each soil layer and estimate ϕ angle from Table 8-3 in Chapter 8.

Nordlund Method Procedure

STEP 2 Determine δ, the friction angle between the pile and soil based on the displaced soil volume, V, and the soil friction angle, ϕ.

a. Compute volume of soil displaced per unit length of pile, V.

b. Enter Figure 9-6 with V and determine δ/ϕ ratio for pile type.

c. Calculate δ from δ/ϕ ratio.

Nordlund Method Procedure

STEP 3 Determine the coefficient of lateral earth pressure K_δ for each soil friction angle, ϕ.

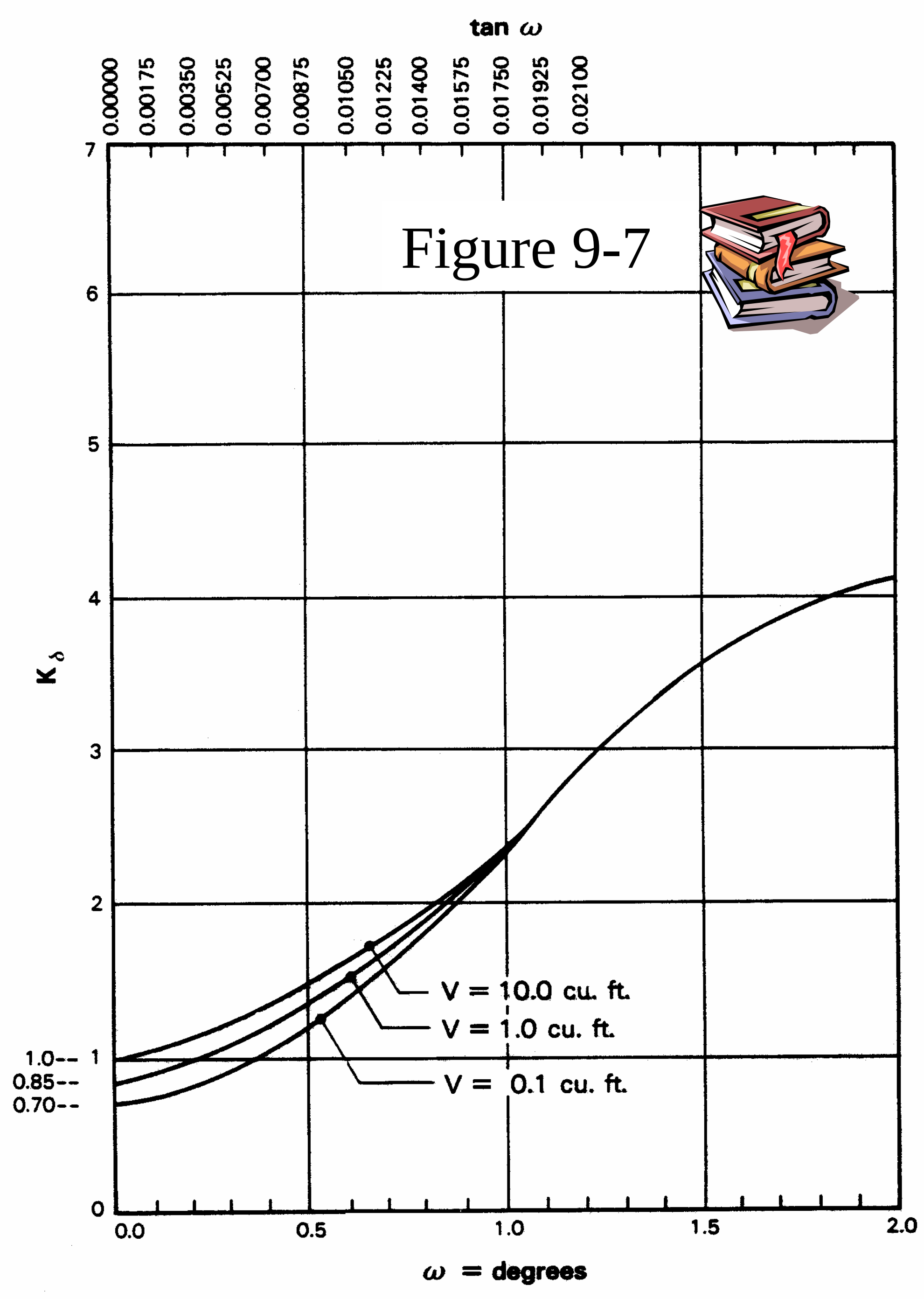
a. Determine K_δ for each ϕ angle based on displaced volume V, and pile taper angle, ω, using appropriate procedure in steps 3b, 3c, 3d, or 3e.

b. If displaced volume is 0.1, 1.0, 10 ft^3/ft and the friction angle is 25, 30, 35, or 40, use Figures 9-7 to 9-10.

c. If displaced volume is given but ϕ angle is not. Linear interpolation is required to determine K_δ for ϕ angle.

K_δ versus ω

$\phi = 25^\circ$



Nordlund Method Procedure

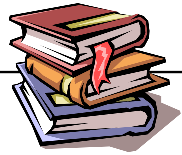
STEP 3 Determine the coefficient of lateral earth pressure K_δ for each soil friction angle, ϕ.

d. If displaced volume is not given but ϕ angle is given, log linear interpolation is required to determine K_δ for displaced volume V.

e. If neither the displaced volume or ϕ angle are given, first use linear interpolation to determine K_δ for ϕ angle and then use log linear interpolation to determine K_δ for the displaced volume, V.

See Table 9-6 for K_δ as function of ϕ angle and displaced volume V

Table 9-6(a) Design Table for Evaluating K_δ for Piles when $\omega = 0^\circ$ and V = 0.10 to 1.00 ft³/ft



ϕ	Displaced Volume -V, ft³/ft									
	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
25	0.70	0.75	0.77	0.79	0.80	0.82	0.83	0.84	0.84	0.85
26	0.73	0.78	0.82	0.84	0.86	0.87	0.88	0.89	0.90	0.91
27	0.76	0.82	0.86	0.89	0.91	0.92	0.94	0.95	0.96	0.97
28	0.79	0.86	0.90	0.93	0.96	0.98	0.99	1.01	1.02	1.03
29	0.82	0.90	0.95	0.98	1.01	1.03	1.05	1.06	1.08	1.09
30	0.85	0.94	0.99	1.03	1.06	1.08	1.10	1.12	1.14	1.15
31	0.91	1.02	1.08	1.13	1.16	1.19	1.21	1.24	1.25	1.27
32	0.97	1.10	1.17	1.22	1.26	1.30	1.32	1.35	1.37	1.39
33	1.03	1.17	1.26	1.32	1.37	1.40	1.44	1.46	1.49	1.51
34	1.09	1.25	1.35	1.42	1.47	1.51	1.55	1.58	1.61	1.63
35	1.15	1.33	1.44	1.51	1.57	1.62	1.66	1.69	1.72	1.75
36	1.26	1.48	1.61	1.71	1.78	1.84	1.89	1.93	1.97	2.00
37	1.37	1.63	1.79	1.90	1.99	2.05	2.11	2.16	2.21	2.25
38	1.48	1.79	1.97	2.09	2.19	2.27	2.34	2.40	2.45	2.50
39	1.59	1.94	2.14	2.29	2.40	2.49	2.57	2.64	2.70	2.75
40	1.70	2.09	2.32	2.48	2.61	2.71	2.80	2.87	2.94	3.0

Nordlund Method Procedure

STEP 4 Determine the correction factor C_F to be applied to K_δ if $\delta \neq \phi$.

Use Figure 9-11 to determine the correction factor for each K_δ.
Enter figure with ϕ angle and δ/ϕ ratio to determine C_F.

Correction Factor for K_δ when $\delta \neq \phi$

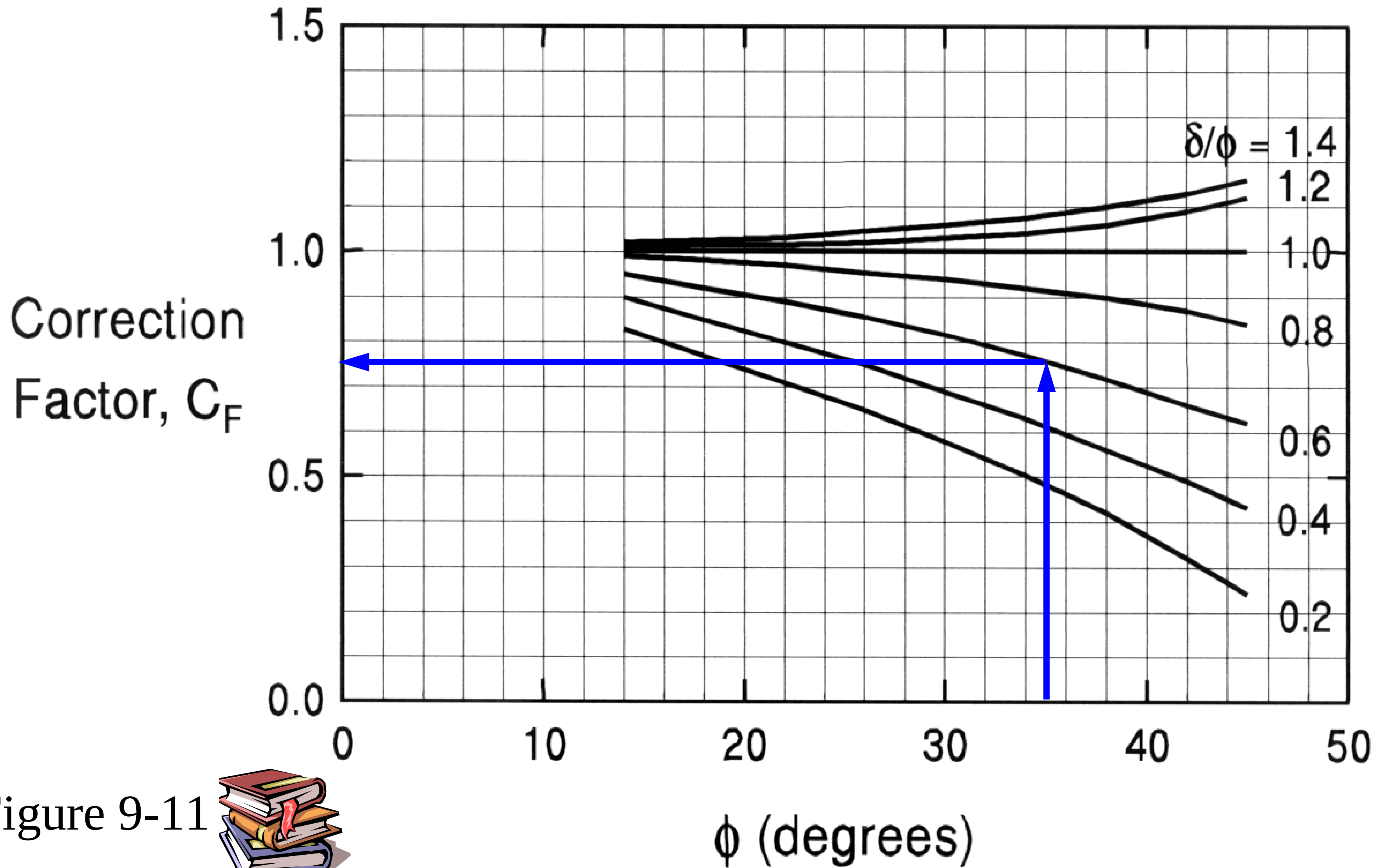


Figure 9-11

Nordlund Method Procedure

STEP 5 Compute the average effective overburden pressure at the midpoint of each soil layer.

STEP 6 Compute the shaft resistance in each soil layer. Sum the shaft from each layer to obtain the ultimate shaft resistance, R_S.

$$R_s = K_\delta \, C_F \, p_d \, \sin\delta \, C_d \, D$$

Nordlund Method Procedure

STEP 7 Determine the α_t coefficient and the bearing capacity factor, N'_q, from the ϕ angle near the pile toe.

a. Enter Figure 9-12(a) with ϕ angle near pile toe to determine α_t coefficient based on pile length to diameter ratio.

b. Enter Figure 9-12(b) with ϕ angle near pile toe to determine, N'_q.

c. If ϕ angle is estimated from SPT data, compute the average corrected SPT $N1_{60}$ value over the zone from the pile toe to 3 diameters below the pile toe. Use this average corrected SPT $N1_{60}$ value to estimate ϕ angle near pile toe from Table 8-3.

Nordlund Method Procedure

STEP 8 Compute the effective overburden pressure at the pile toe.

NOTE: The limiting value of p_t is 3 ksf (150 kPa)

STEP 9 Compute the ultimate toe resistance, R_t.

Use lesser of:

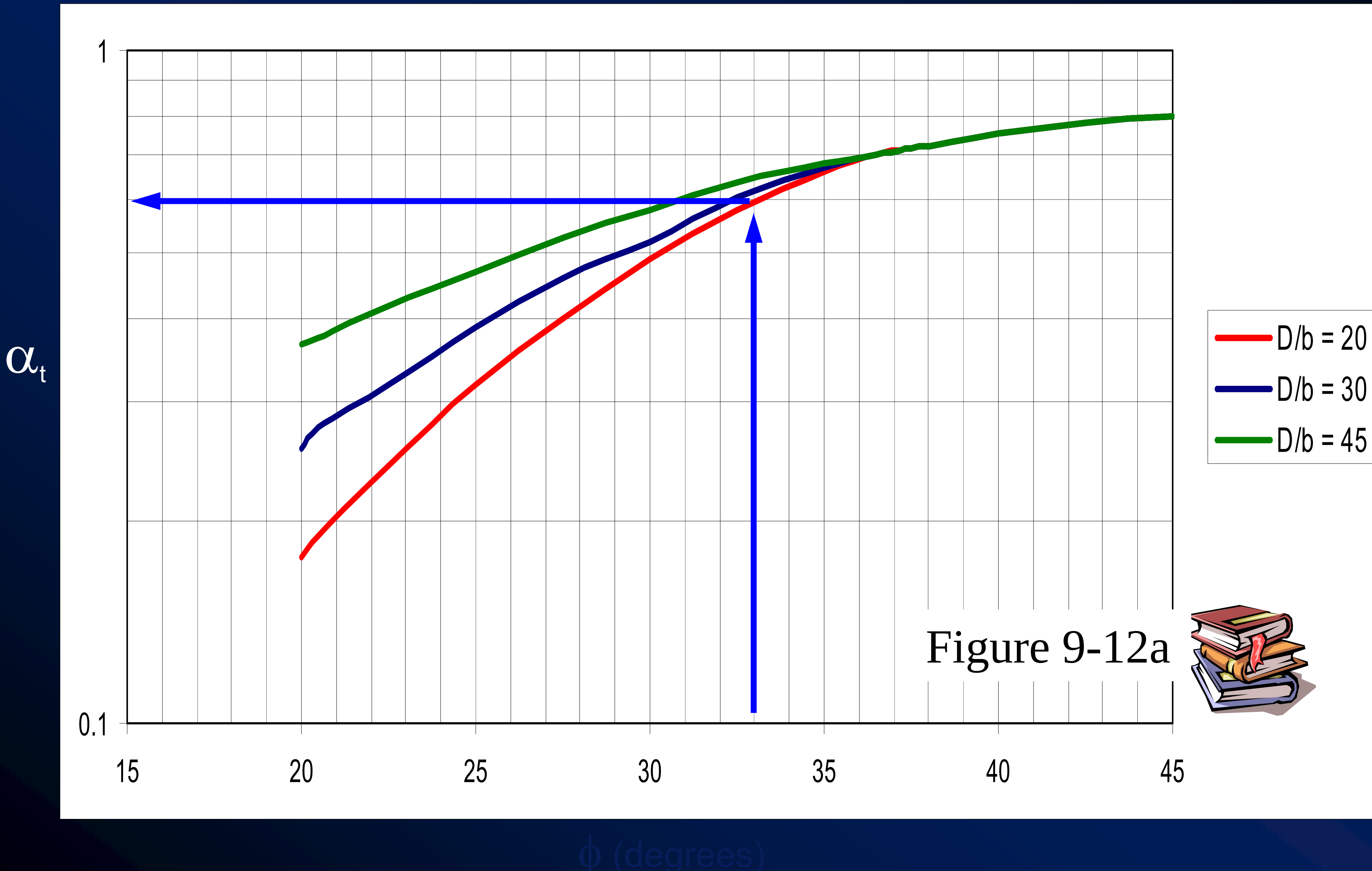
$$R_t = \alpha_t N'_q p_t A_t$$

Figure 9-12a and 9-12b

$$R_t = q_L A_t$$

Figure 9-13

α_t Coefficient versus ϕ



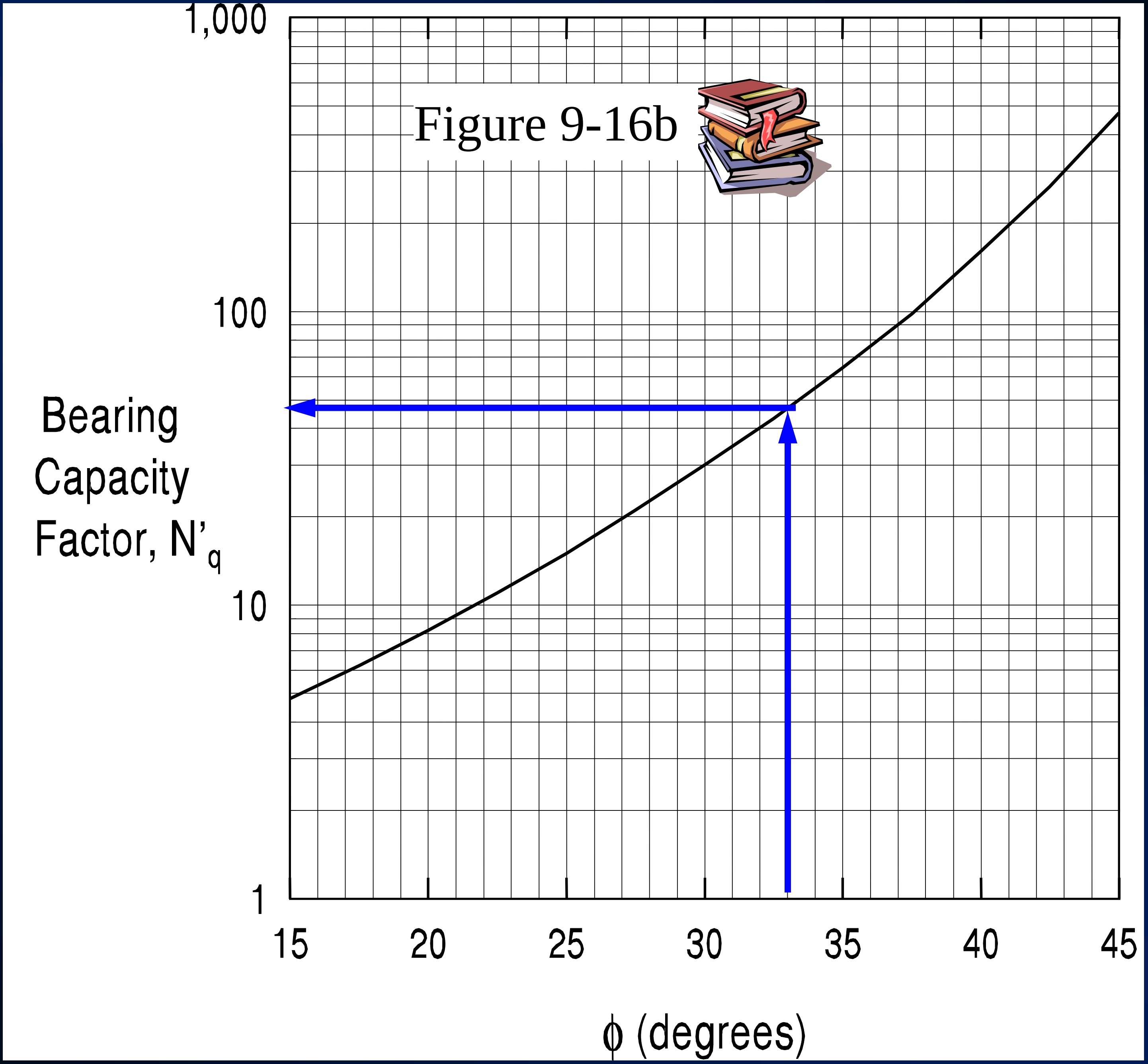


Figure 9-16b
1,000
100
10
1
Bearing
Capacity
Factor, N'q
15
20
25
30
35
40
45
φ (degrees)

Limiting Unit Toe Resistance