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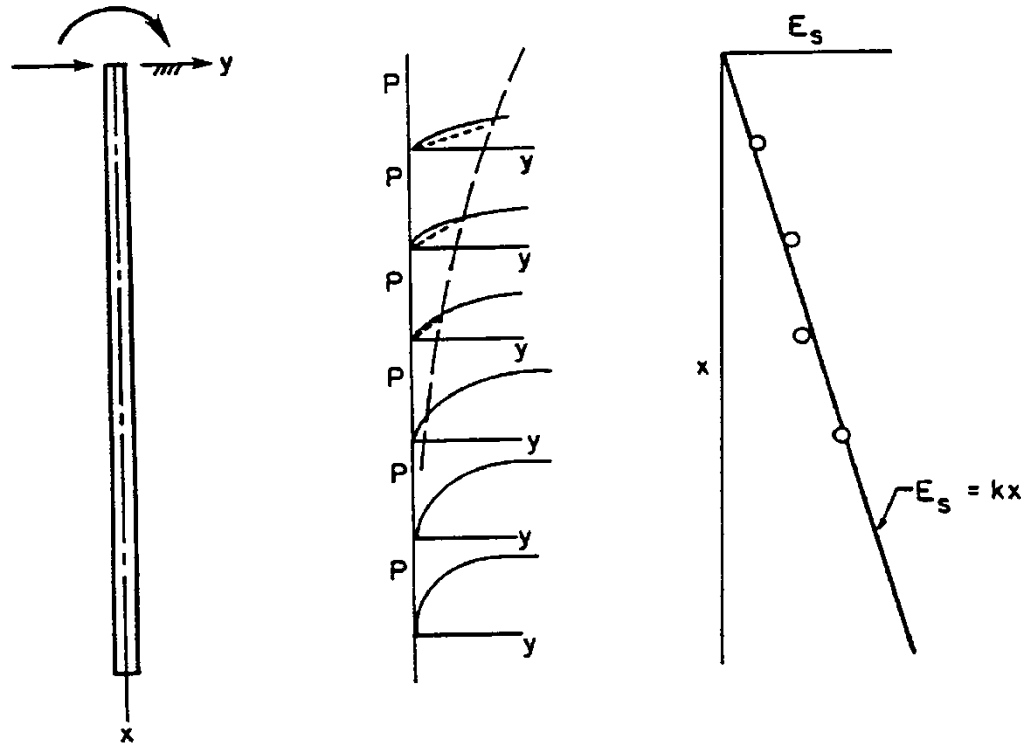


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ENCE 4610

Foundation Analysis and Design



(a) Pile

(b) p-y curves

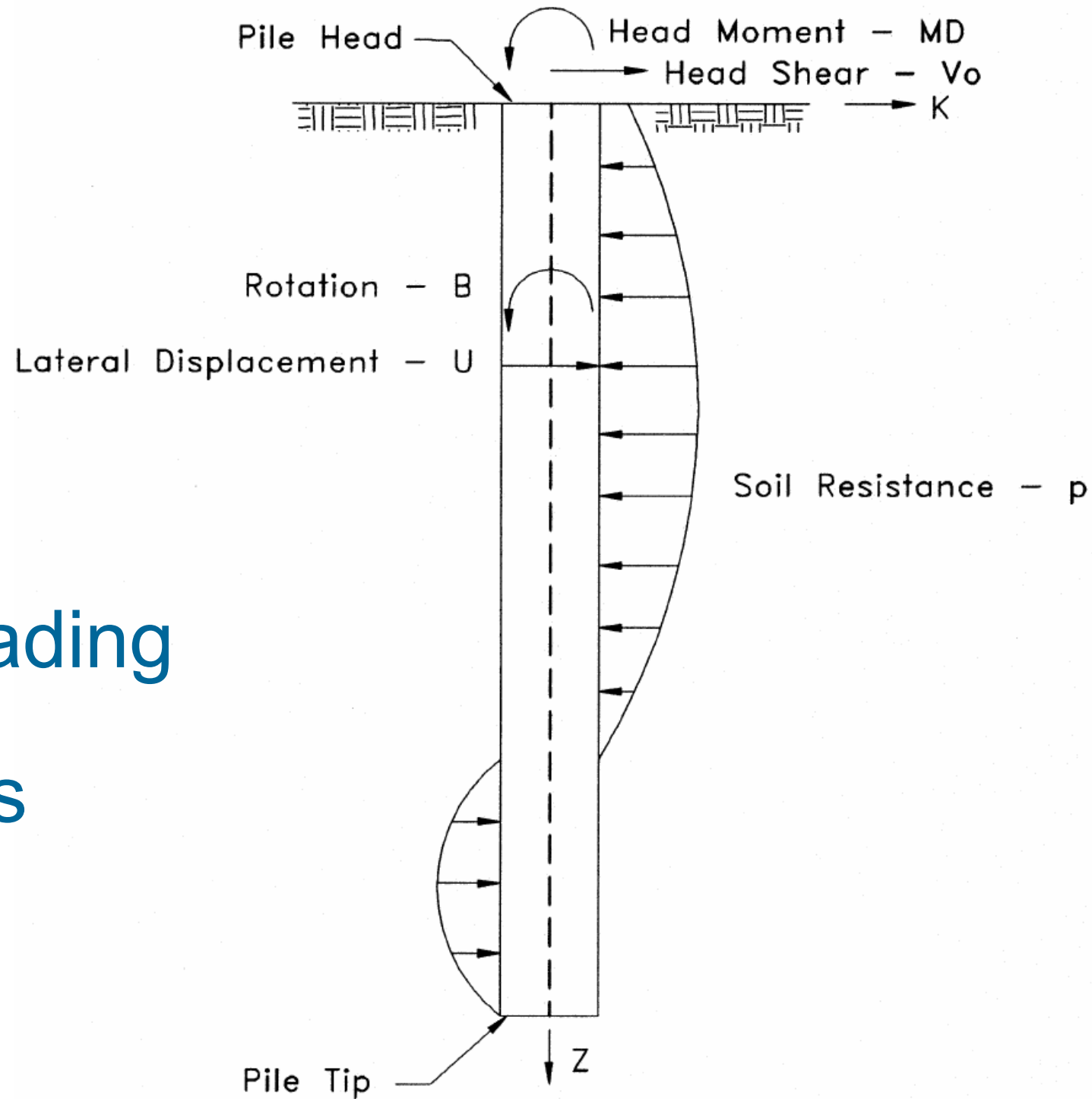
(c) Soil modulus

Lateral Loading of Piles
Drilled Shafts and Auger-Cast Piles

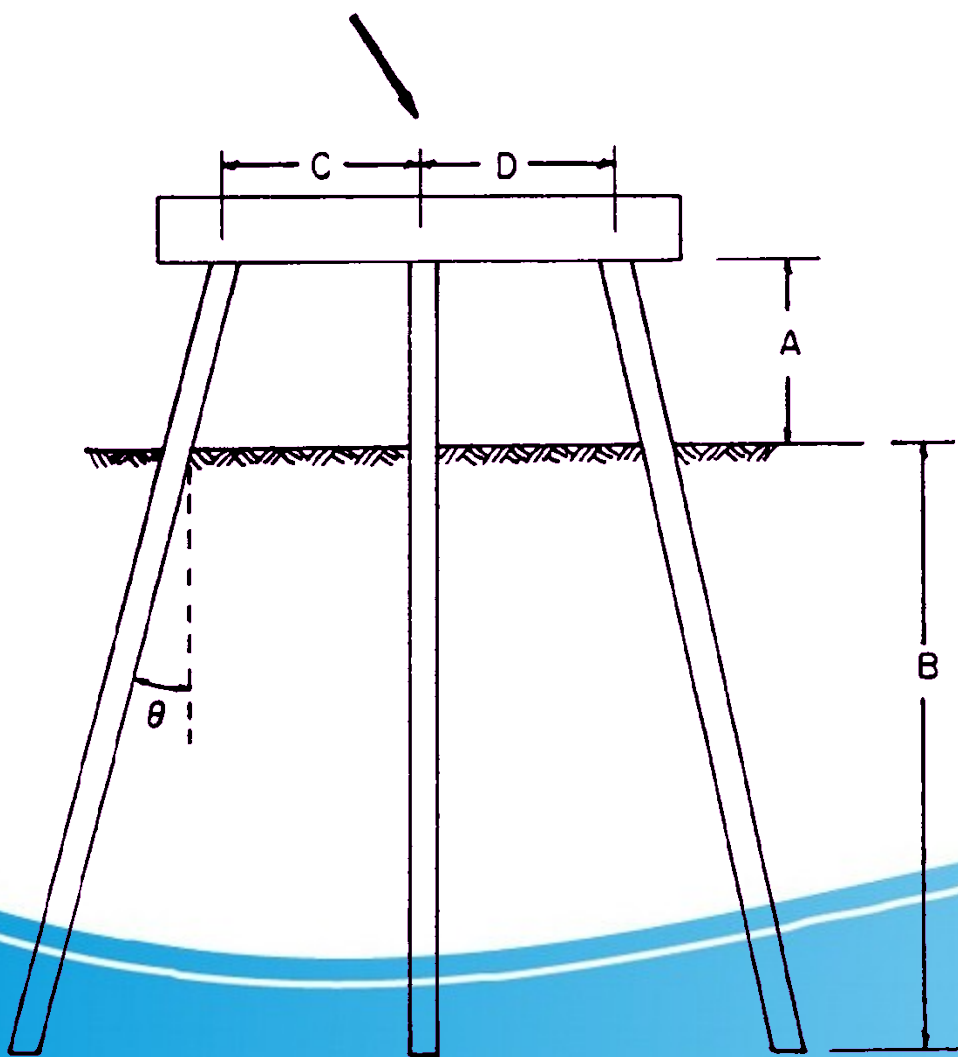
Sources of Lateral Loading

- River current and mud movement loads in alluvial settings (foundations subject to scour)
- Ocean wave forces
- Slope movements
- Cable forces on transmission towers
- Earth pressures on retaining walls
- Wind Loads
- Seismic Loads
- Impact Loads from Ships (Berthing, Pier Collision, etc.)
- Eccentric Loads on Columns

Lateral Loading in Piles



Batter Piles



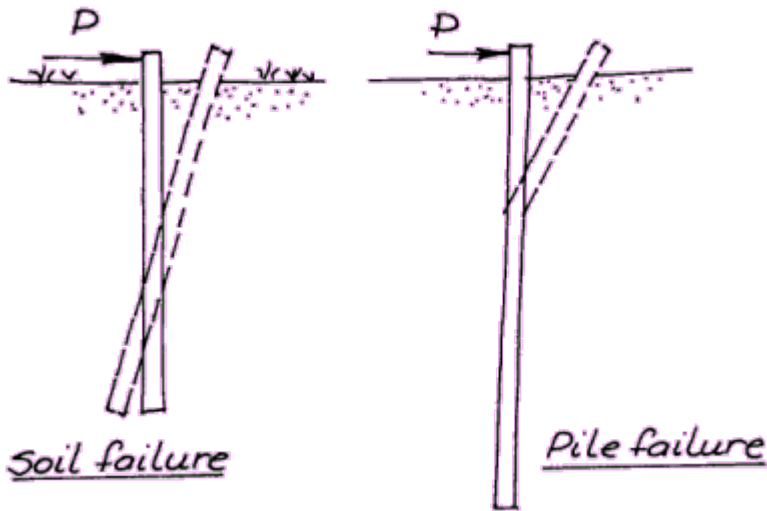
- Basically turn lateral loads into axial loads
- Present challenges in driving and testing
- Form a very stiff system than can pose problems in seismic situations
- Very common solution to lateral loading

Analytic Methods for Lateral Loading

Dividing Line:

Timber – $D/B = 20$

Steel or Concrete – $D/B = 35$

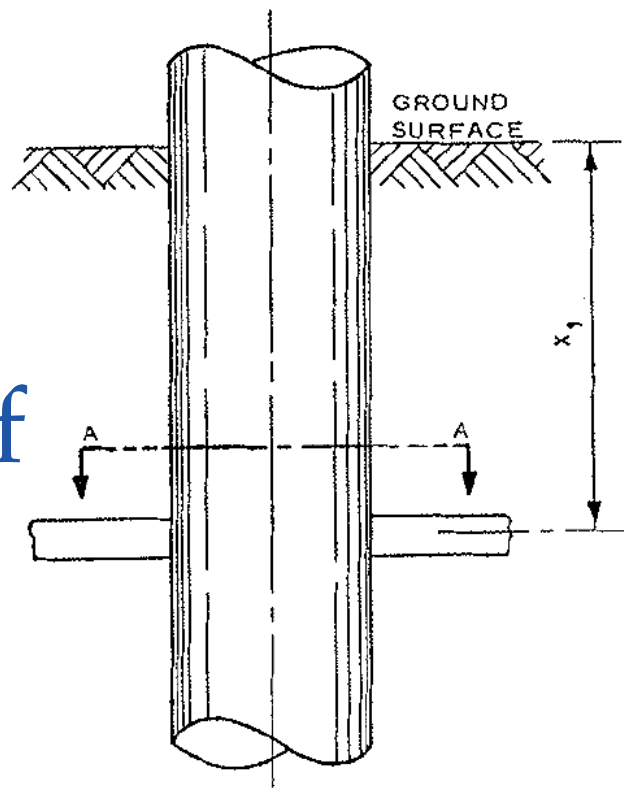


Short Foundations
(Rigid)

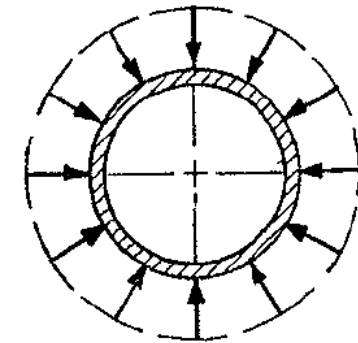
Long Foundations
(Beam-Like)

- Rigid Methods (Broms)
 - Used for light weight « short » foundations
 - Same limitations as rigid methods for mat foundations
- Depth to Fixity Methods (Davisson)
 - Only considers a certain depth as flexible
 - Structural engineers could analyse the foundation as a structure once the depth of fixity was known
 - Too simplistic
- Finite Element Analysis
- p-y curves

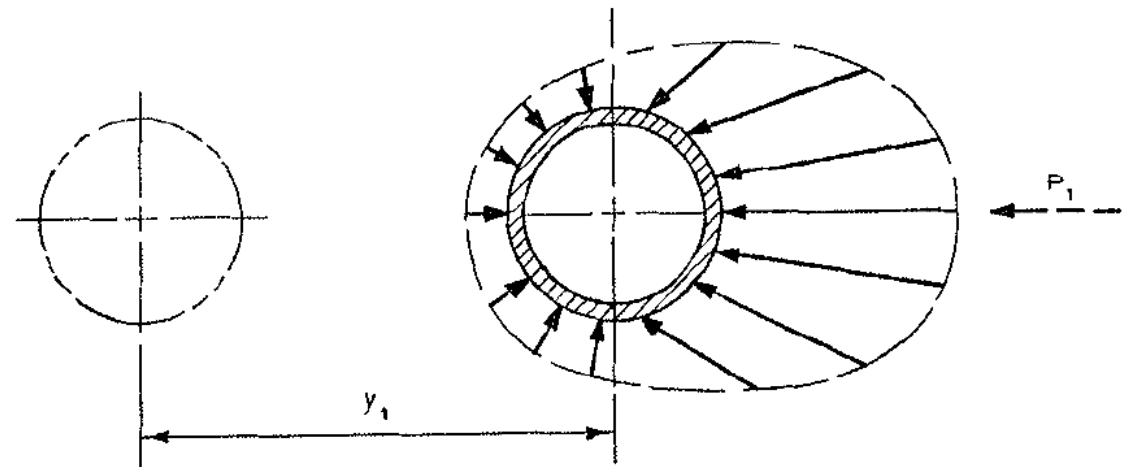
Compression of Soil in Lateral Loading



a. REPRESENTATION OF PILE SEGMENT



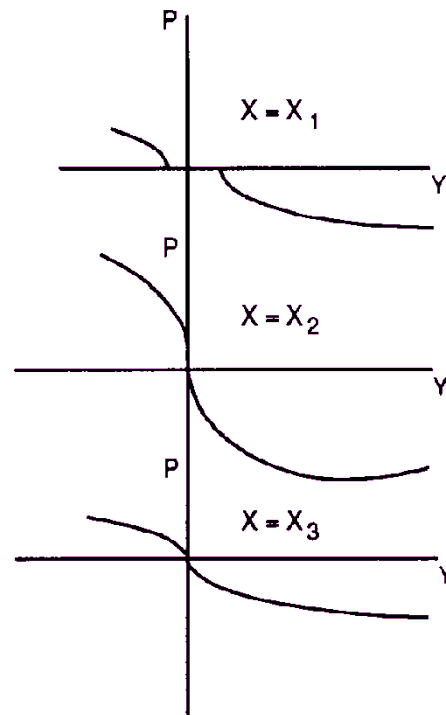
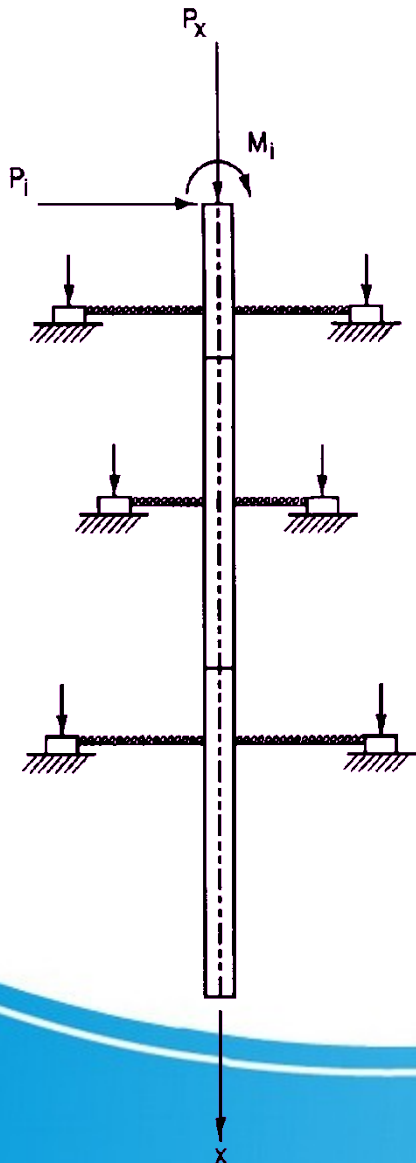
b. PRESSURE DISTRIBUTION
BEFORE LOADING



c. PRESSURE DISTRIBUTION AFTER LOADING

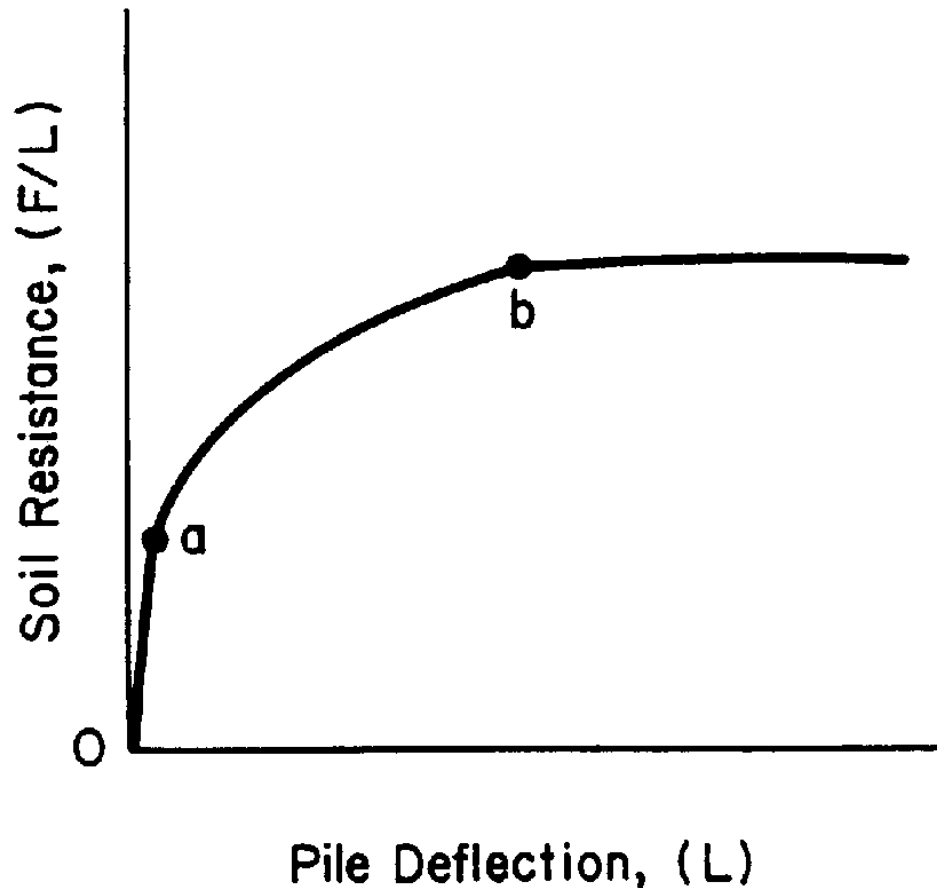
Suction on the load side
Additional stress on the far side

p-y Curves



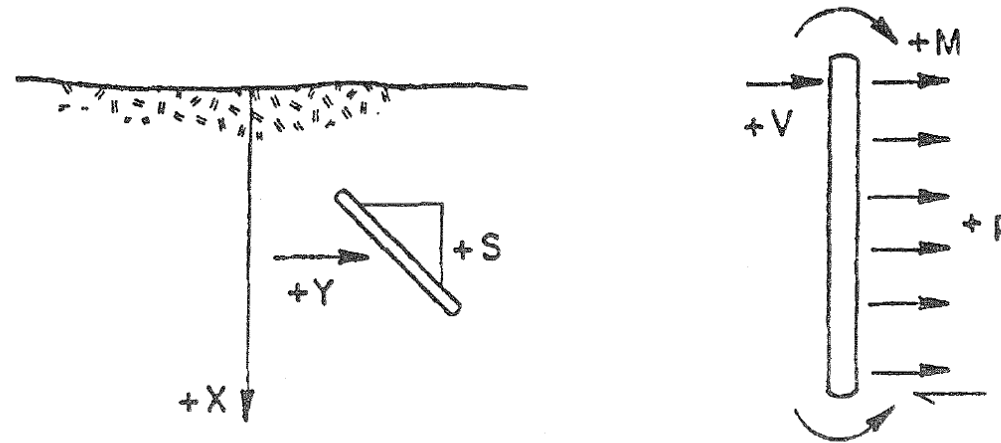
- Take into consideration nonlinear soil characteristics (as opposed to Winkler model)
- Properly require a finite-difference (COM624, LPILE) computer solution
- Non-dimensional and spreadsheet solutions available for common problems

Development of p-y Curves

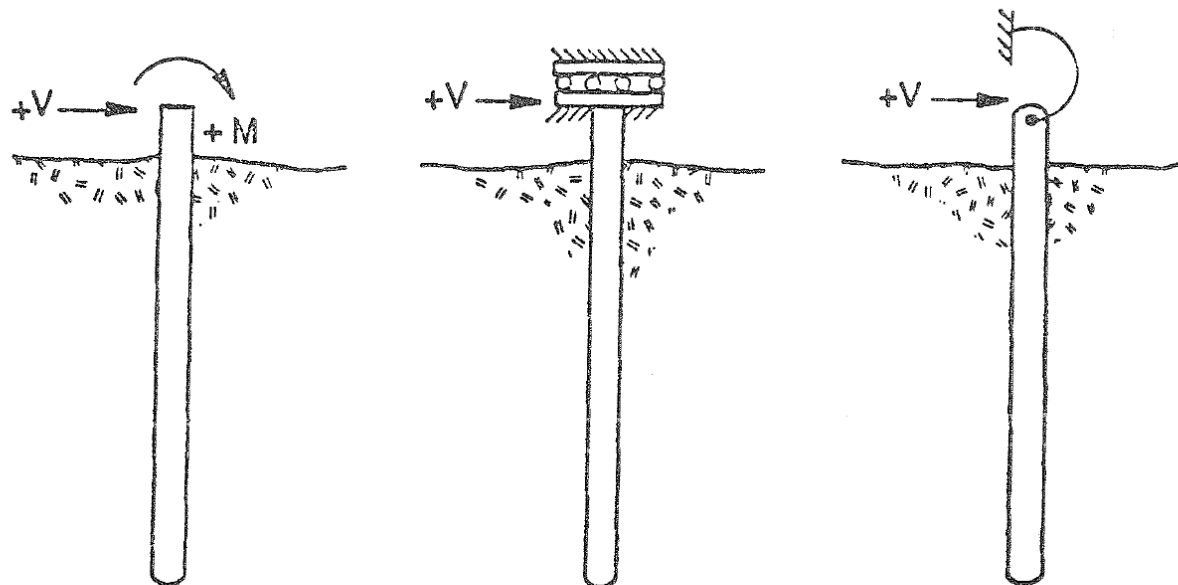


- Empirical Data
 - Based on actual lateral load tests, either on the job site itself or on controlled field tests
- Computer Programs
 - Model the lateral deflection of the pile as a function of depth
 - Take into consideration non-linear soil response
 - Can be difficult to use
- Non dimensional methods based on computer results or empirical data
 - Not as accurate as program, but suitable for estimates or smaller projects

End Fixity for Lateral Loading



a. Sign convention



Case I. Pile Head
Free to Rotate

Case II. Pile Head
Fixed Against Rotation

Case III. Pile Head
Restricted Against Rotation

b. Boundary conditions

Characteristic Load Method (CLM)

- Based on the COM624G program
- Reduce the variables to nondimensional form
- Should be used for preliminary estimates ONLY and not for final design, due to limitations of its underlying assumptions
- Advantages
 - Analyses can be done quickly and simply
 - Can determine load-deflection characteristics directly
- Assumptions
 - Constant EI , s_u or ϕ , and γ for depth of pile
 - Foundation is long enough to be considered fixed at the toe ("long foundation" criterion)
- Three ways of accessing the method
 - Hand calculations, which involve charts and can be tedious
 - CLM2 Spreadsheet, which can analyze both bored and driven piles and pile groups
 - TAMWAVE program, which is easier to input but only valid for single driven pile analysis
- Group Effects
 - Group effects are important with laterally loaded piles as they are with axial ones
 - The effect is usually called the PSPI (pile-soil-pile-interaction), or shadow effect
 - The soil stress created by lateral loads around one pile will extend to the pile's neighbours, depending upon the distance between the piles and the level of stress
 - The lateral capacity of each pile is generally degraded by this effect
 - Methods of solution use p-y curve methods and consider spacings and pile deflections
 - CLM 2.0 Spreadsheet includes group effect calculations

Characteristics Loads and Moments

Basic equations (from Brettmann and Duncan (1996))

Pile group efficiency factors were incorporated in the Characteristic Load Method of analysis by reducing the soil resistance in the expressions for characteristic load (P_c) and characteristic moment (M_c). The undrained strength of clay (S_u), and the coefficient of passive earth pressure for sand (K_p) in those expressions were multiplied by F_m , leading to the following expressions for P_c and M_c .

For clay

$$P_c = 7.34D^2(E_p R_I) \left(\frac{S_u F_m}{E_p R_I} \right)^{0.68} \quad (1)$$

$$M_c = 3.86D^2(E_p R_I) \left(\frac{S_u F_m}{E_p R_I} \right)^{0.46} \quad (2)$$

For sand

$$P_c = 1.57D^2(E_p R_I) \left(\frac{\gamma' D \phi' K_p F_m}{E_p R_I} \right)^{0.57} \quad (3)$$

$$M_c = 1.33D^2(E_p R_I) \left(\frac{\gamma' D \phi' K_p F_m}{E_p R_I} \right)^{0.40} \quad (4)$$

where, P_c = characteristic load (force units, F), M_c = characteristic moment (force times length, FL), D = pile or shaft width or diameter (L), E_p = pile or drilled shaft modulus of elasticity (F/L^2), R_I = moment of inertia ratio (dimensionless), S_u = undrained shear strength for clay soil (F/L^2), F_m = pile or drilled shaft group efficiency based on pile spacing (dimensionless), γ' = effective unit weight for sand (F/L^3), ϕ' = effective stress friction angle of sand (degrees), K_p = Rankine coefficient of passive earth pressure of sand (dimensionless). Any consistent set of units may be used.

The value of R_I is the ratio of the moment of inertia of the pile to the moment of inertia of a solid circular section of the same width, or diameter.

Evans and Duncan

Example

INPUT PARAMETERS			
UNITS			
	FORCE =	KIPS	
	LENGTH =	IN	
SOIL PROPERTIES			
	$\phi =$	36	DEGREES
	$\gamma =$	6.94E-05	KIPS/IN ³
	$K_p =$	3.85	
PILE PROPERTIES			
	$D_o =$	12	IN
	$D_i =$	0	IN
	$I =$	1728	IN ⁴
	$R_{cr} =$	1	
	$E_p =$	4400	KIPS/IN ²
	$L =$	720	IN
PILE GROUP PROPERTIES			
	$N_{row} =$	30	

- Given
 - 12" Square Concrete Pile
 - Restrained head
 - 60' long, 12" square
 - $f'_c = 6000$ psi
 - Shear load = 20 kips
 - Soil: Sand, $\phi' = 36$ deg., $\gamma = 120$ pcf
 - Groundwater table at depth of 40'
- Find
 - Lateral deflection of the pile top
 - Maximum moment at pile top

Evans and Duncan

Example (Spreadsheet)

FIXED HEAD				
			SINGLE PILE	
Pt	Pt/Pc	Yt/D	Yp	Mmax
KIPS			IN	IN-KIPS
per Pile				
10.0	0.0033	0.0054	0.065	317.0
20.0	0.0065	0.0152	0.183	780.5
30.0	0.0098	0.0280	0.336	1322.2
40.0	0.0131	0.0431	0.517	1921.9
50.0	0.0164	0.0603	0.723	2568.7
60.0	0.0196	0.0792	0.950	3255.7
70.0	0.0229	0.0998	1.198	3978.1
80.0	0.0262	0.1219	1.463	4732.2
90.0	0.0294	0.1455	1.746	5515.2
100.0	0.0327	0.1704	2.045	6324.8

Evans and Duncan Example (TAMWAVE)

- Answers are slightly different between the two because TAMWAVE uses a “default” value for concrete E which is different from the CLM2 spreadsheet
- Input required modification of default soil properties

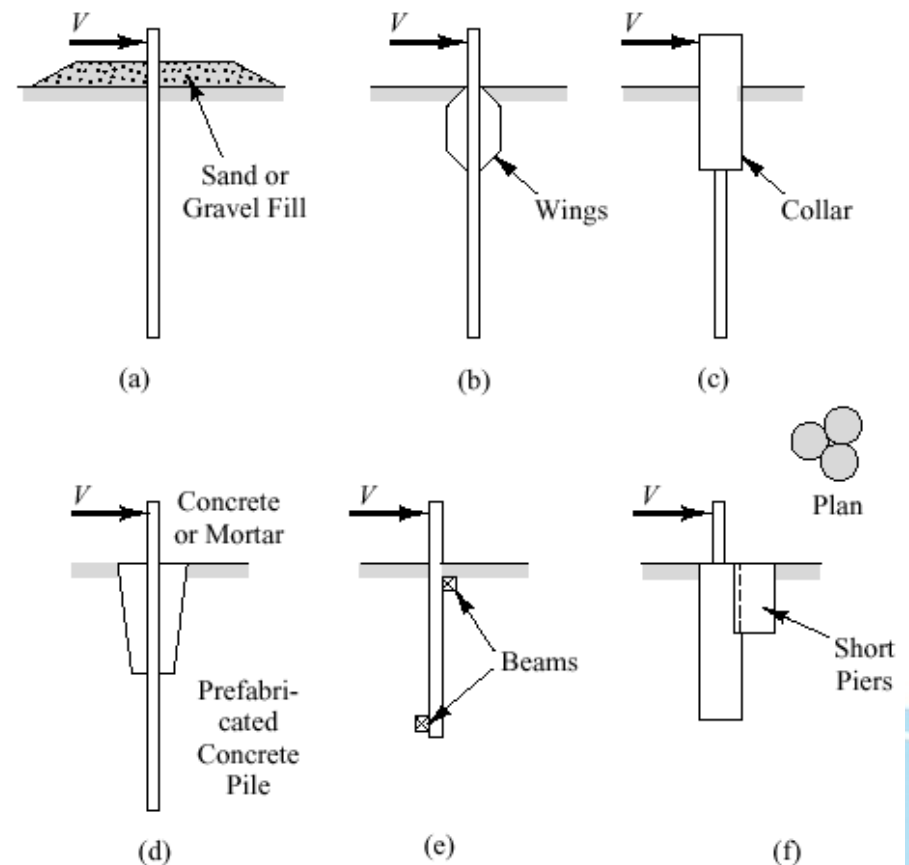
Data for Lateral Load Analysis using CLM2 Method	
Nominal Soil Unit Weight, lb/in ³	0.06944
Pile Moment of Inertia, in ⁴	1,728.00
Pile Section Modulus, in ³	288.00
Pile Solid Circle Moment of Inertia, in ⁴	1,017.88
Moment of Inertia Ratio R_i	1.698
Pile Moment of Inertia Ratio Product, ksi	8,488.3
Pile-Soil Interaction Variable	73,456
Pile L/D Ratio	60.0
Characteristic Load, lbs.	3,231,798.4
Characteristic Moment, in-lbs.	220,700,080.8
Pile Head Fixity	Fixed
Pile Head Lateral Load, lbs.	20,000.0
P_t/P_c	0.00619
Y_t/D	0.01402
Pile Head Deflection due to Load, inches	0.168
Maximum Moment Due to Pile Head Lateral Load, in-lbs	784,274.1
Maximum Bending Stress Due to Pile Head Lateral Load, in-lbs	2,723.2

Lateral Load Verification and Enhancement

Verification

- Full-scale lateral load tests
 - As with axial tests, slow and expensive, but the best way to determine lateral load capacity
 - Always a reaction test
 - Can be used to back calculate p-y curves
- Model lateral load tests
 - Conditions are controlled, but extrapolation is difficult
- Lateral Static Tests
 - Only used as an impact load test

Enhancement



ENCE 4610

Foundation Analysis and Design

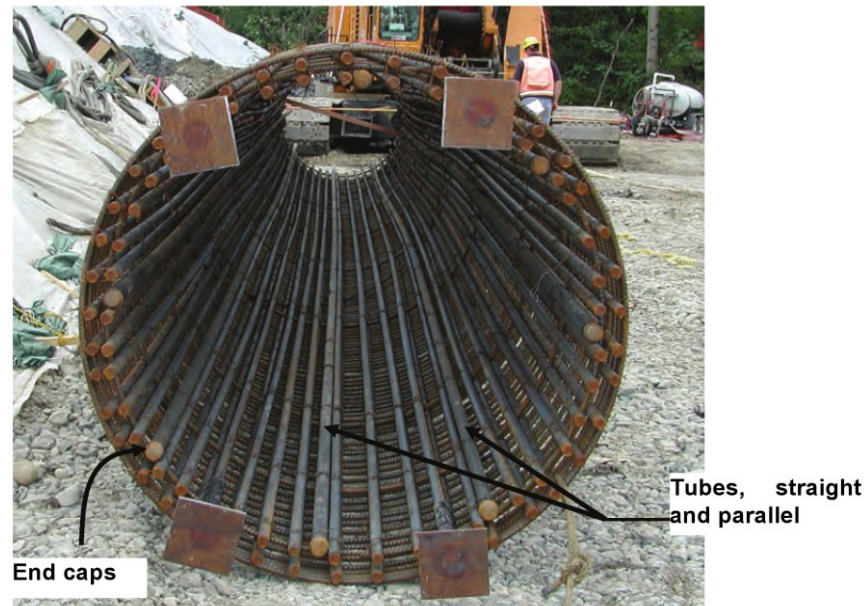
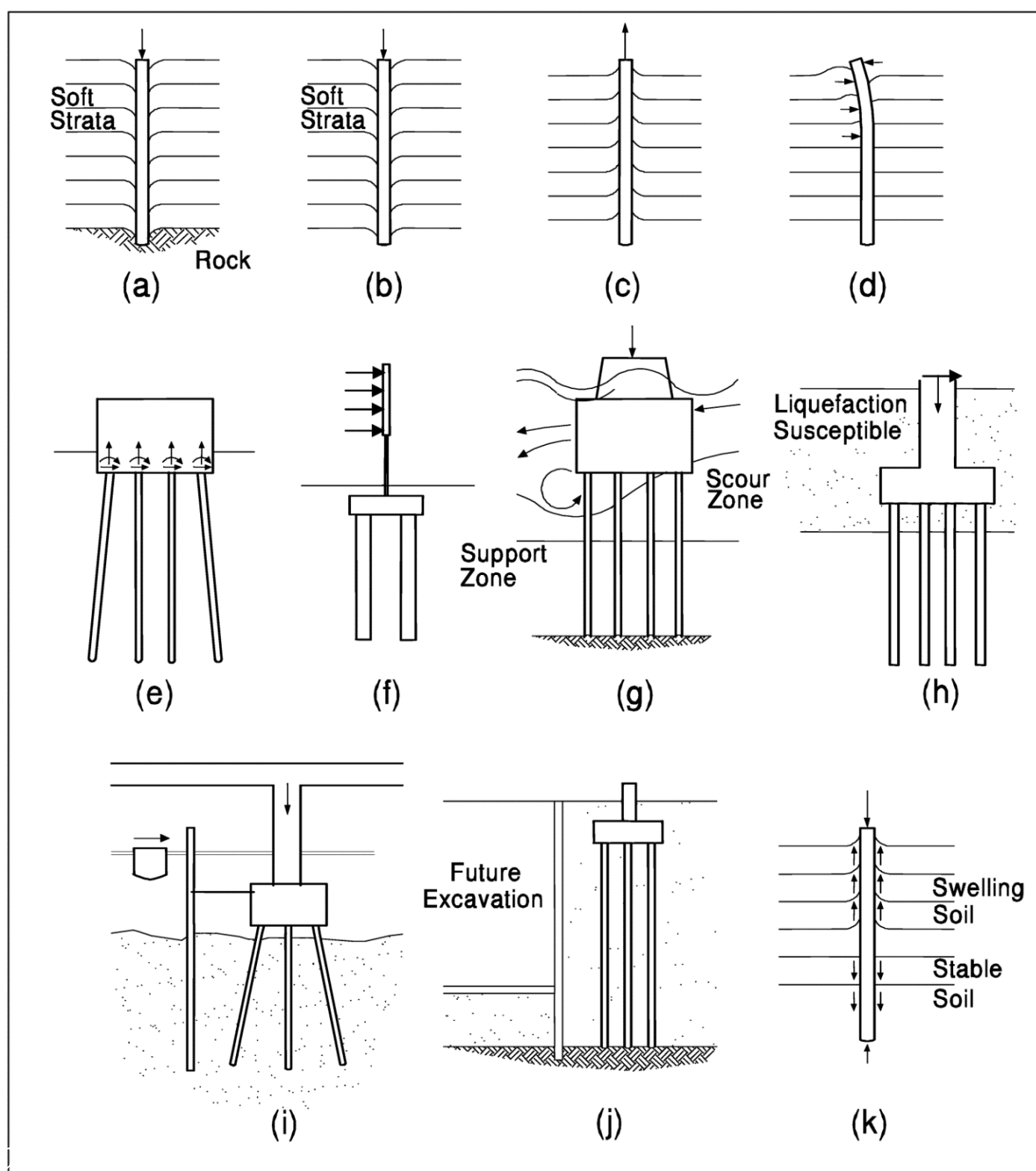


Figure 20-3 Reinforcing Cage with Steel Access Tubes for CSL Testing

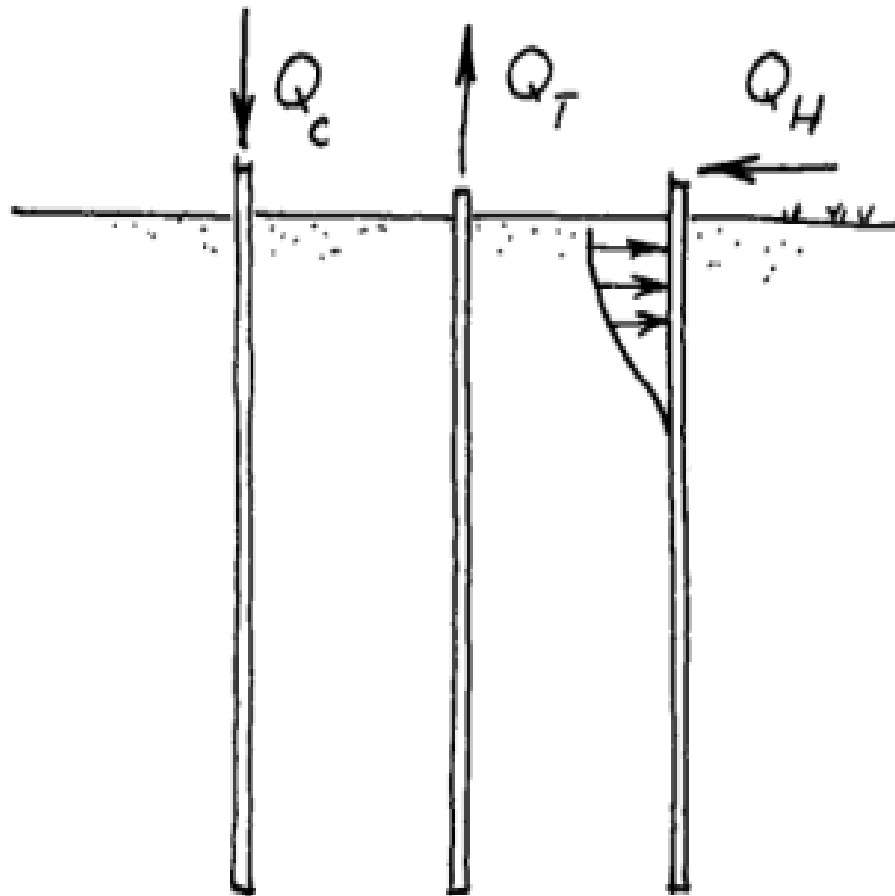
Drilled Shafts and Auger Cast Piles

Reasons for Deep Foundations

Also: For



Loading of Deep Foundations



(a) Compression

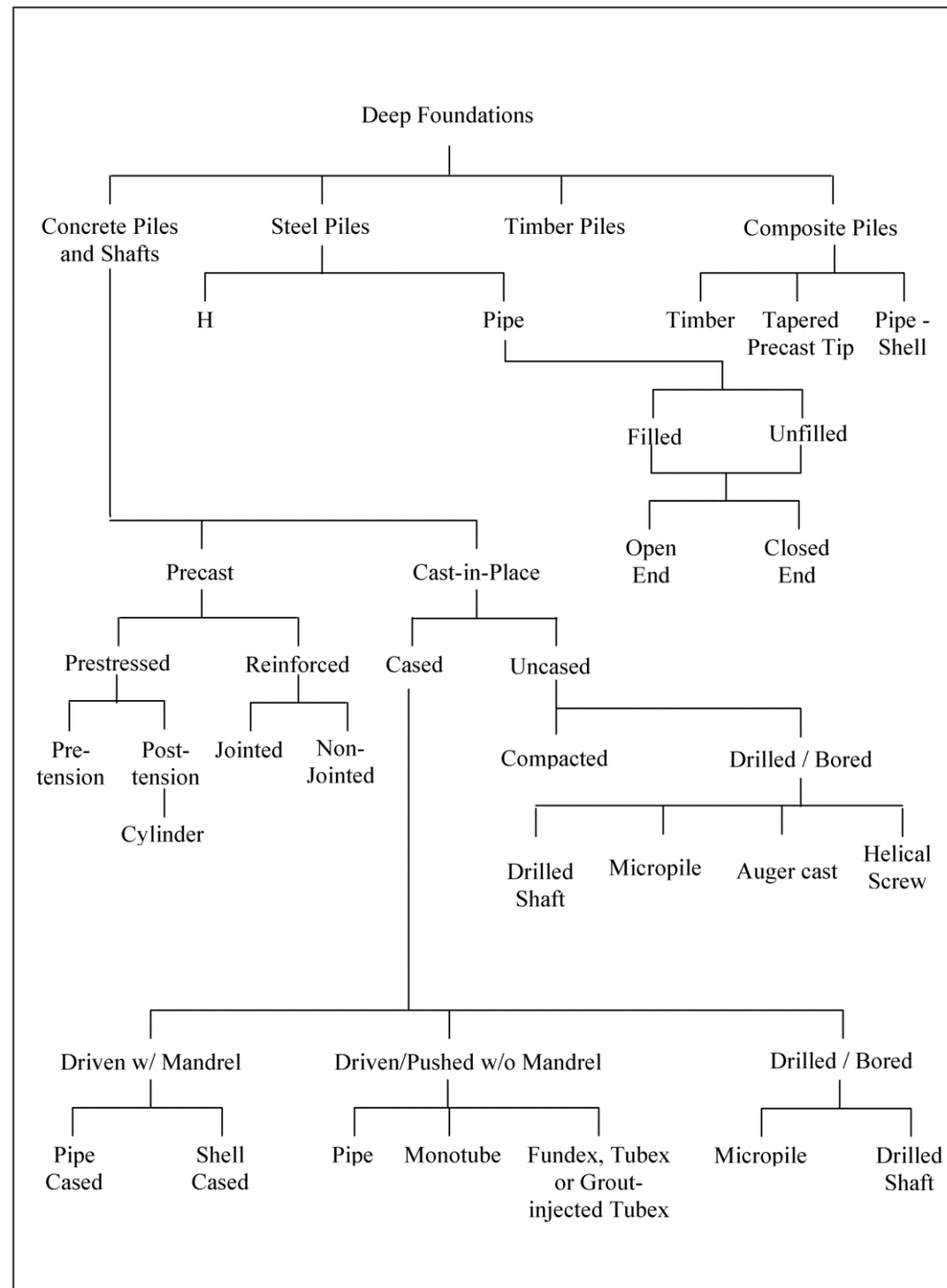
(b) Tension

(c) Lateral load

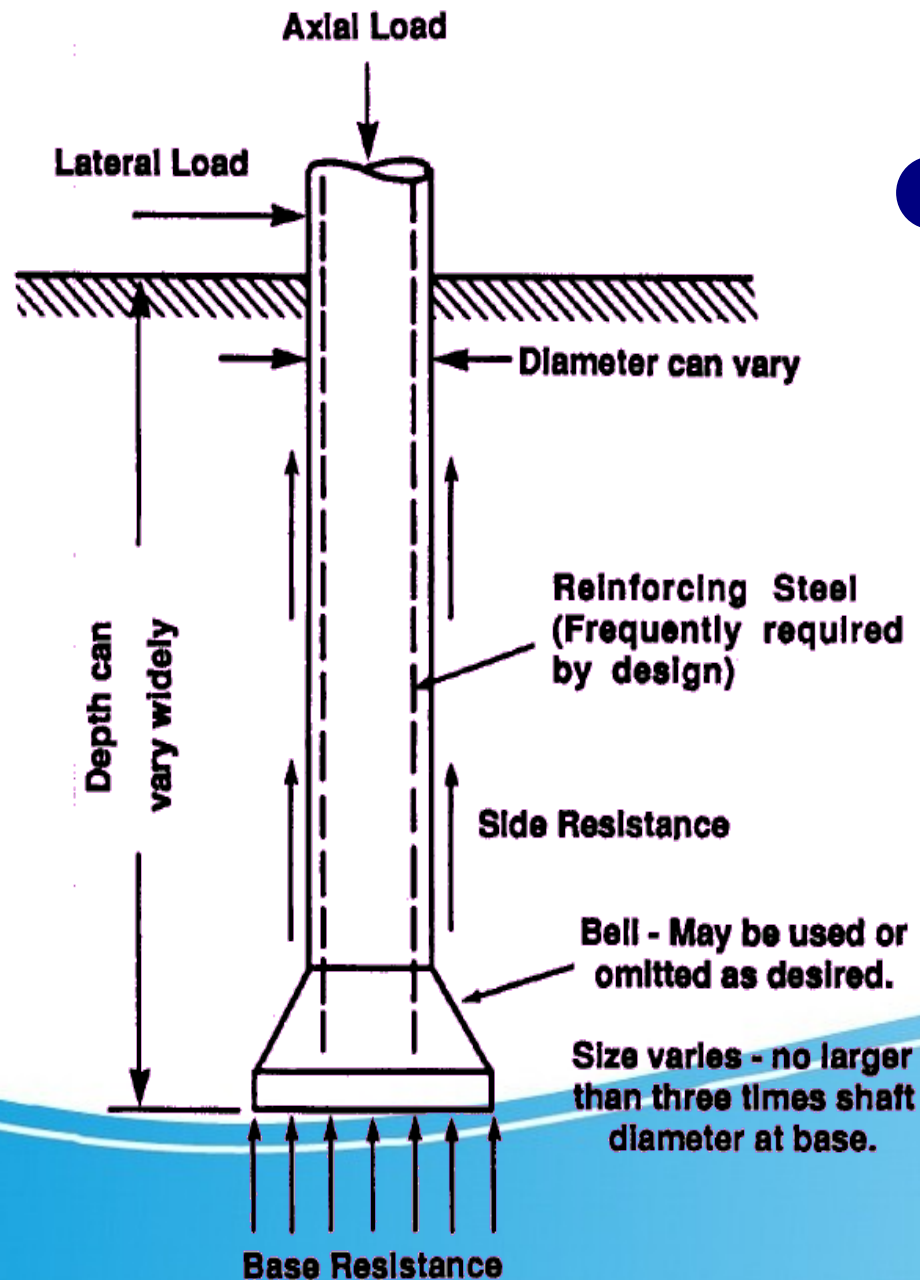
- One of the main reasons for deep foundations is the ability of deep foundations to bear loads that shallow foundations cannot
 - Lateral Loads
 - Tension Loads
 - Compression Loads in Soft Soils



Types of Deep Foundations



Drilled Shafts



Other terms

- Pier
- Drilled pier
- Bored pile
- Cast-in-place piles
- Caisson
- Drilled Caisson
- Cast-in-drilled-hole (CIDH) foundation

History of Drilled Shafts

- Started as an extension of shallow foundations; until 1920's most were hand dug
 - Chicago Well Method used a wooden "barrel" form to prevent collapse of the soil
- Drilling rigs first used in states with oil drilling, whose technology was applied to shaft drilling
- Technology of cutting tools, casing and drilling mud, then advanced on its own
- Drilled shafts are widely used today in a wide variety of geographical areas

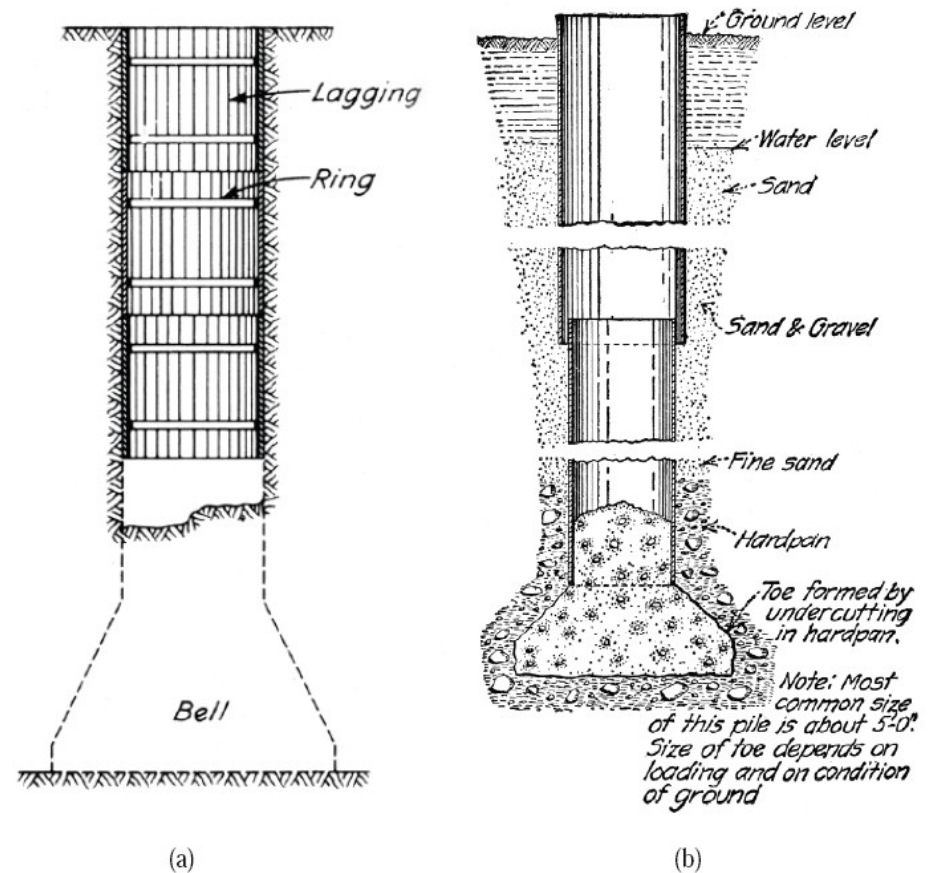


Figure 1-6 Early "Caisson" Foundations (Rogers, 2006); (a) "Chicago Method" and (b) "Gow Caisson"

Equipment for Drilled Shafts

- Most drilled shafts are 500 – 1200 mm (18"-48") in diameter and 6 – 24 m (20'-80') in length
- Most are drilled with a truck mounted rig; specialised rigs are used for longer and larger shafts
- The flight auger used is a helix-shaped drill bit with a relatively short flight
- For rock, hardened teeth can be added to the end of the auger to enhance rock drilling capabilities

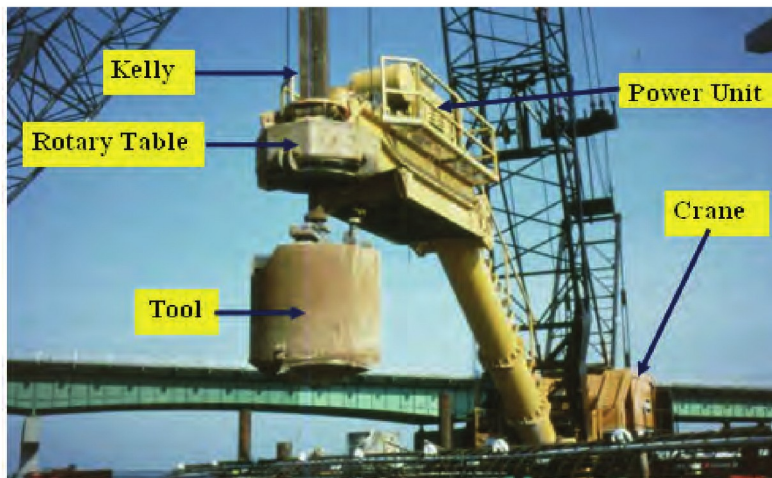


Figure 5-2

Drill Rig Terminology

Advantages and Disadvantages of Drilled Shafts

- Disadvantages

- Successful construction depends upon the quality control and skill the contractor can exercise on the pour; defects are usually not visible and can be serious
- Driving piles pushes soil aside (cavity expansion,) thus increasing capacity of piles of same size (both shaft and toe)
- Full scale load testing is mandatory to determine actual load bearing capacity of drilled shafts; this is being mitigated by dynamic (CAPWAP) and semi-dynamic (Statnamic) methods

- Advantages

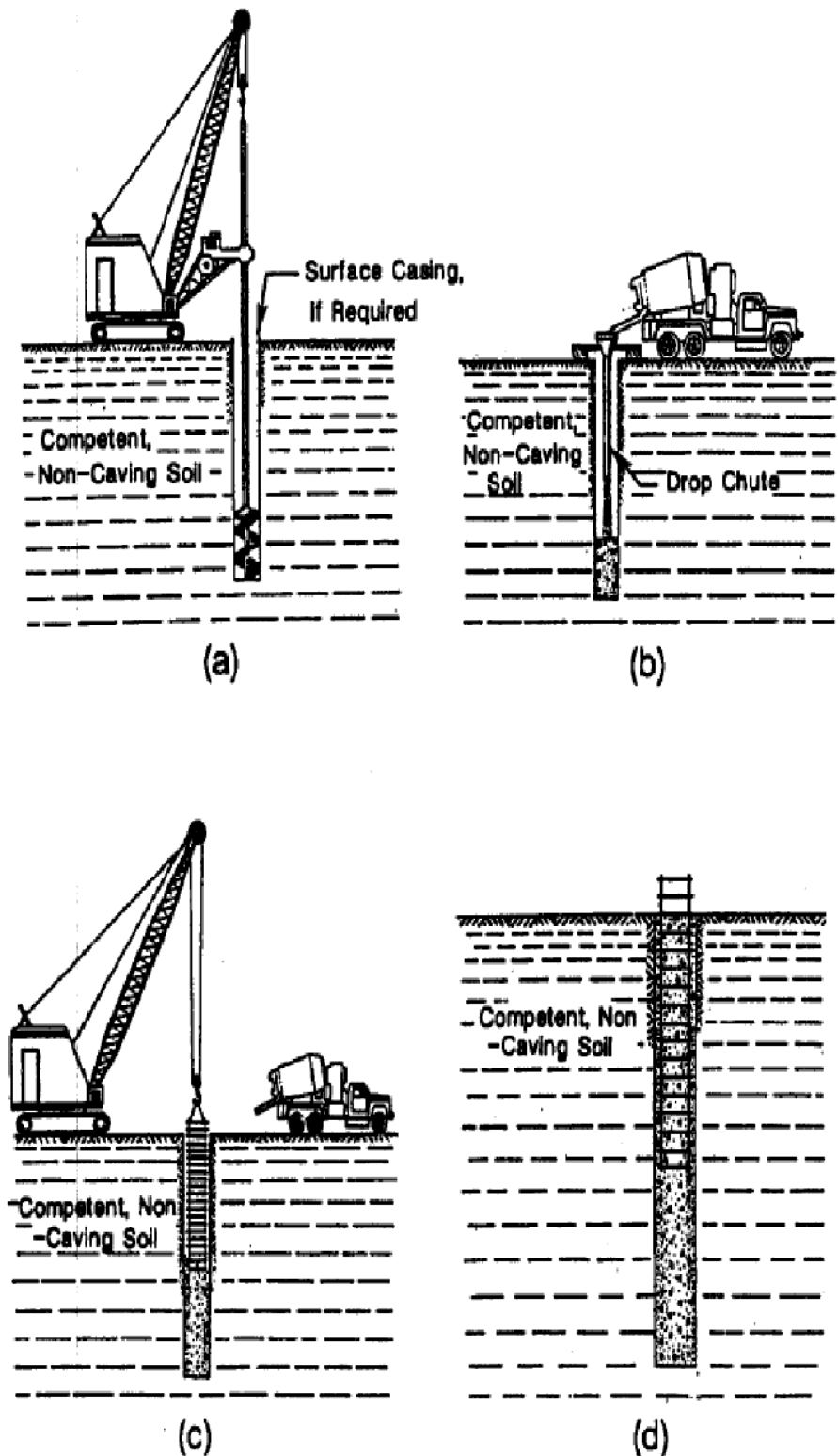
- Mobilisation and demobilisation costs are in many cases lower
- Construction process generates less noise and vibration
- Diameter and length of the shaft can be changed during the job more easily
- With proper cutting tools, can penetrate through cobbles and boulders and even in to rock
- Shafts can be large and columns can be supported with one shaft as opposed to multiple driven piles

Drilled Shafts in Firm Soils

- Dry method (preferable) is preferred method to install a drilled shaft
 - Only applicable to competent, non-caving soils
- Procedure
 - Using a drill rig, excavate the shaft to the required depth
 - Fill the lower end of the shaft with concrete
 - Place a prefabricated reinforcing steel cage inside of the shaft
 - Fill the shaft with concrete



Figure 4-1 Dry Hole in Stable Soil



Drilled Shafts in Firm Soils

- Firm soils offer a relatively straightforward material into which to drill a shaft
- Holes are usually drilled without any special support
- After the holes are drilled, the reinforcing cage is inserted and the hole filled with concrete
- Caving is usually not a problem in firm soils
- If drilling under the groundwater table, water is pumped out as the hole advances and concrete is placed in dewatered shaft



Figure 1-10 Construction of Drilled Shaft in Dry, Cohesive Soils

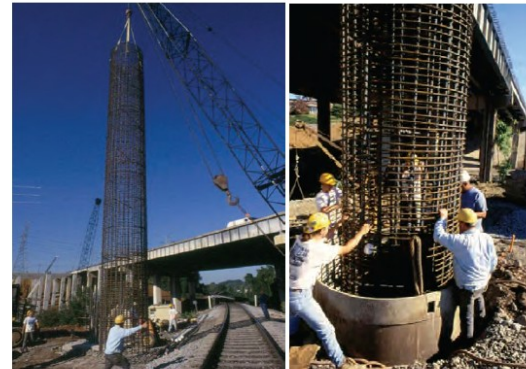
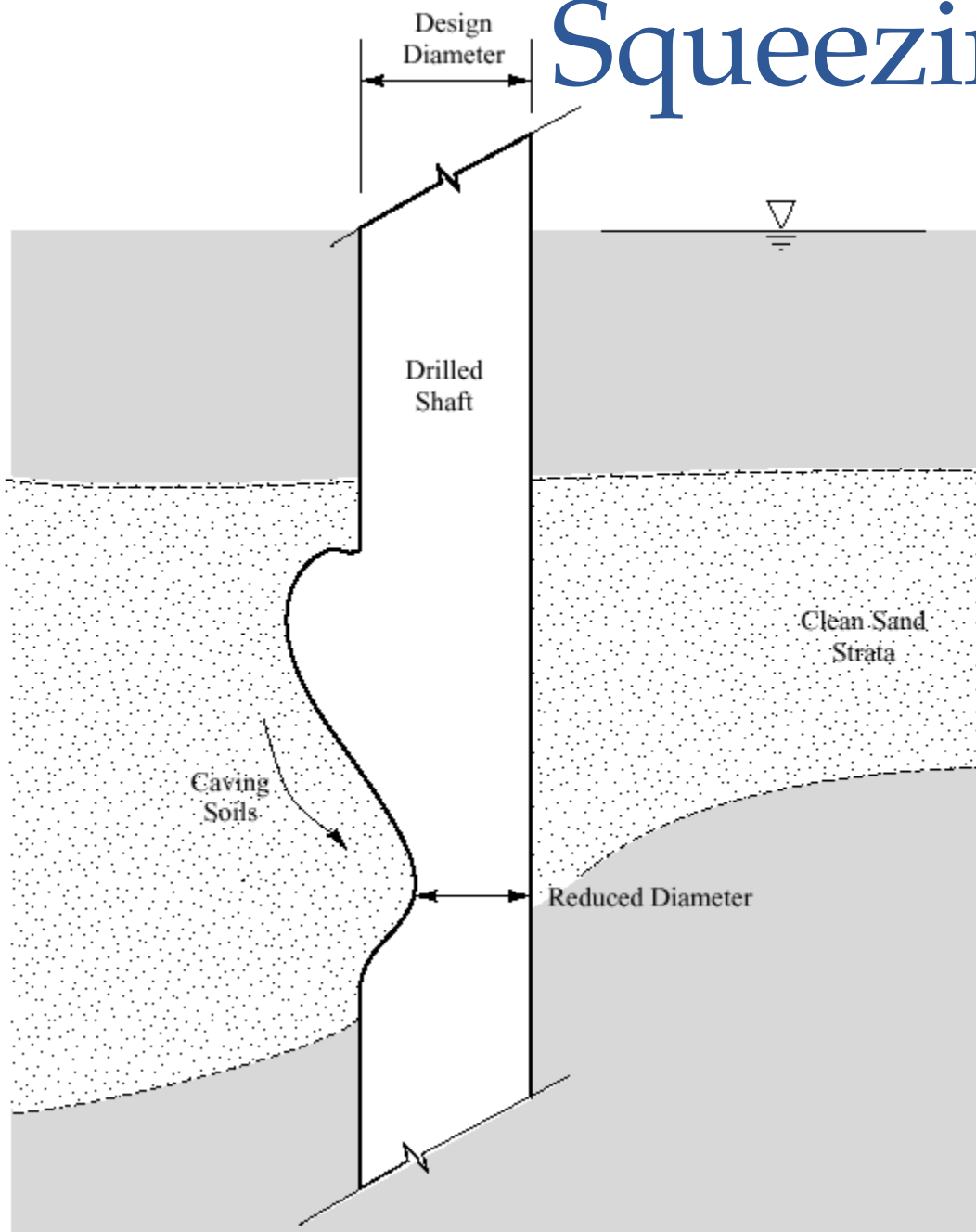


Figure 1-11 Drilled Shafts for Bridge Foundations where Small Footprint is Desirable

Drilled Shafts in Caving or Squeezing Soils



- Soft or caving soils present serious problems for conventional drilled shafts
- Solutions
 - Slurry Method
 - Casing Method

Slurry and Casing Methods

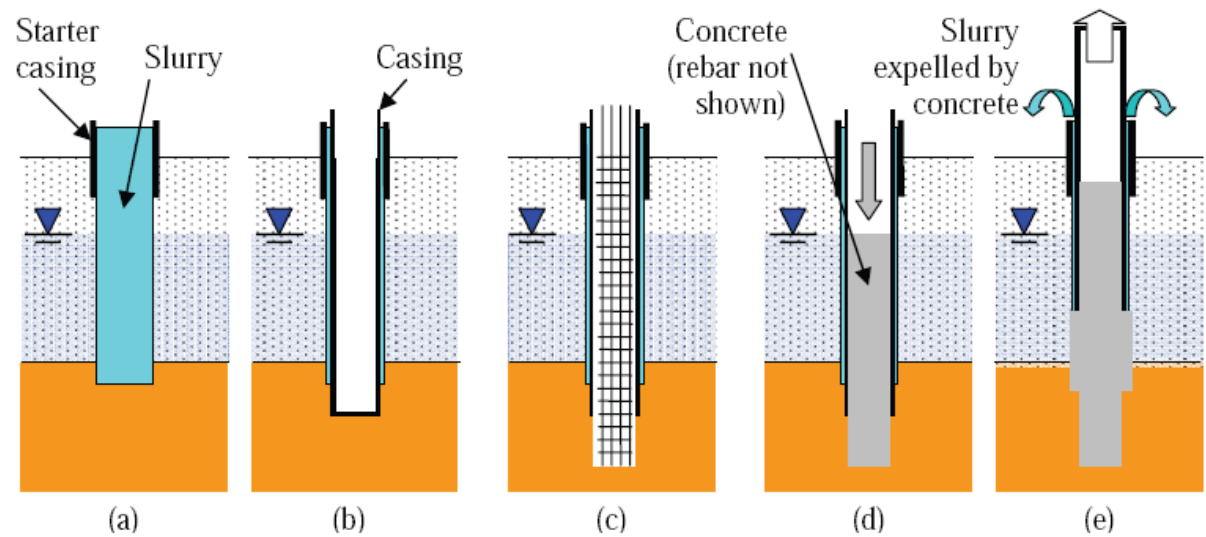


Figure 4-7 Construction Using Casing Through Slurry-Filled Starter Hole: (a) drill with slurry; (b) set casing and bail slurry; (c) complete and clean excavation, set reinforcing; (d) place concrete to head greater than external water pressure; (e) pull casing while adding concrete

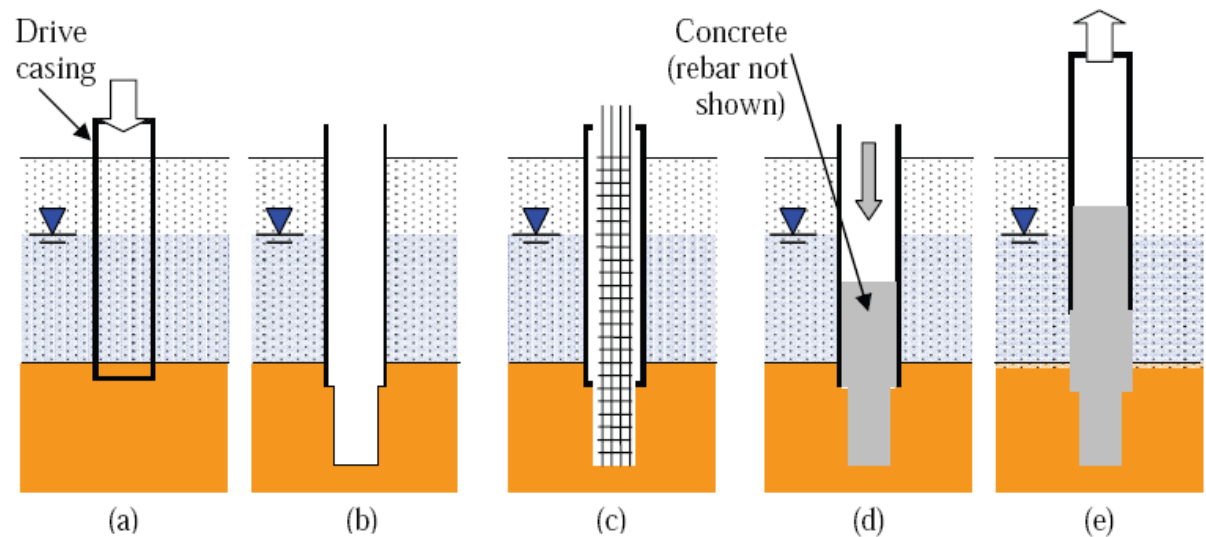


Figure 4-8 Construction Using Casing Advanced Ahead of Excavation: (a) drive casing into bearing stratum; (b) drill through casing; (c) complete and clean hole, set reinforcing; (d) place concrete to head greater than external water pressure; (e) pull casing while adding concrete

Use and Methods of Advancing Casing



Figure 4-6 Drilling into Rock through a Cased Hole



Figure 4-9 Oscillator Rig Used to Advance Segmental Casing Ahead of the Excavation



Figure 4-10 Use of a Vibro-Hammer (Left) and Twister Bar (Right) to Advance Casing

Results of Poor Excavation and Casing



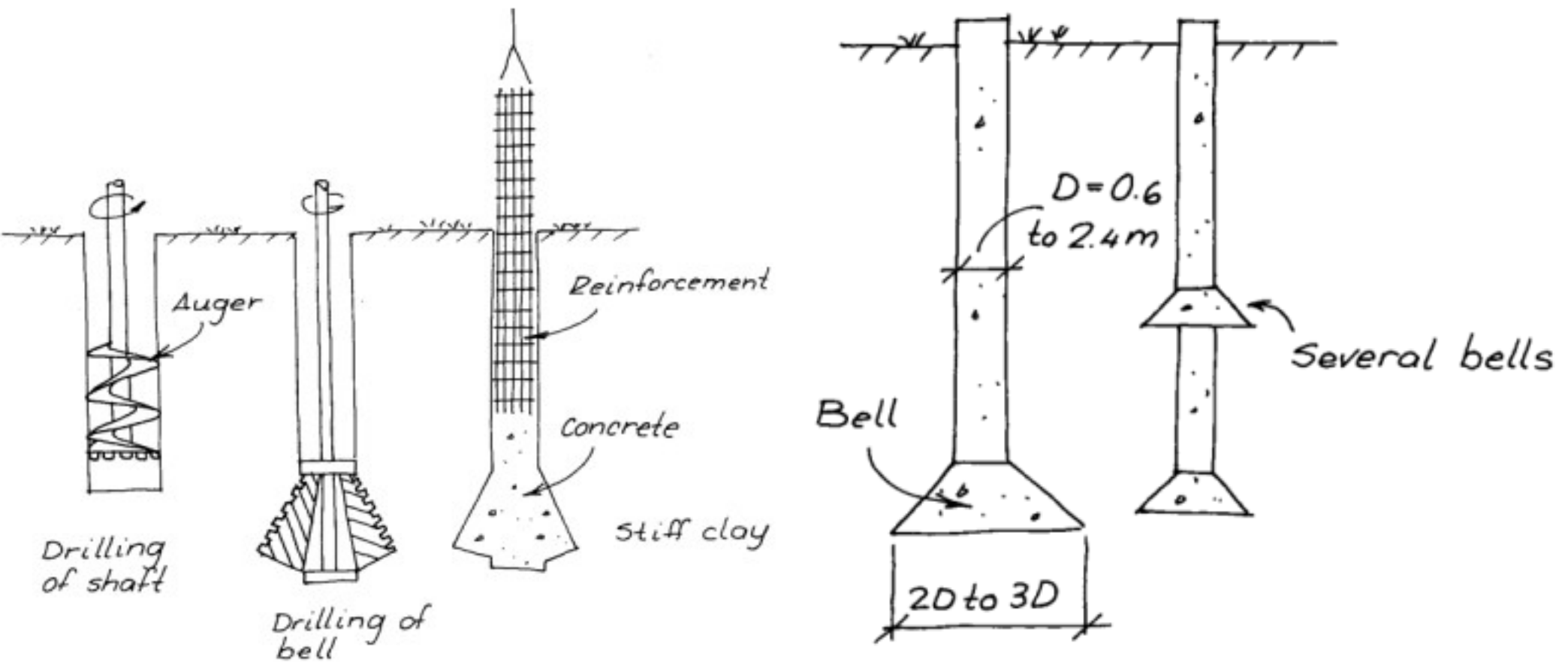
(a)



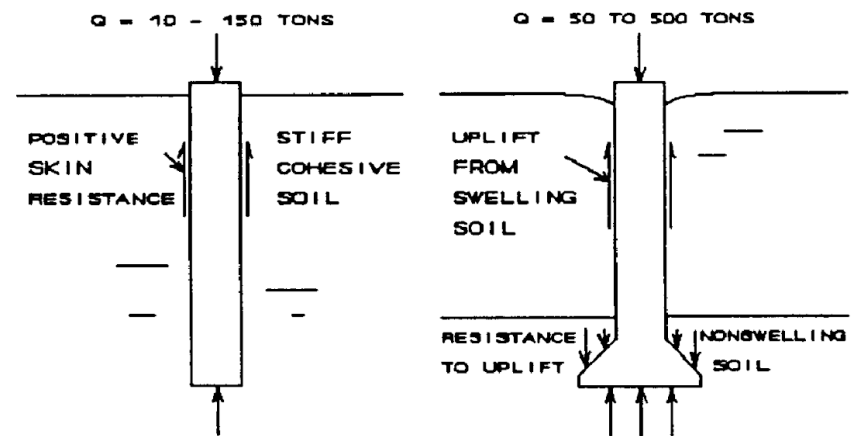
(b)

Figure 9-57. Photographs of exhumed shafts (a) shaft where excavation was not adequately cleaned, (b) shaft where excavation was properly cleaned (FHWA, 2002d).

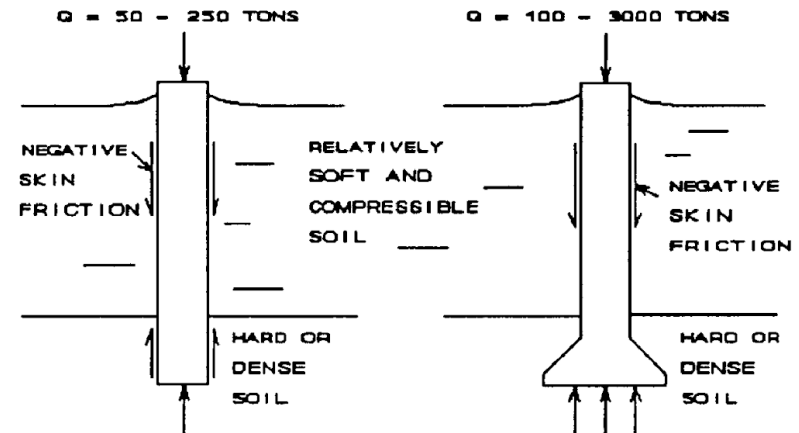
Underreamed (Belled) Shaft



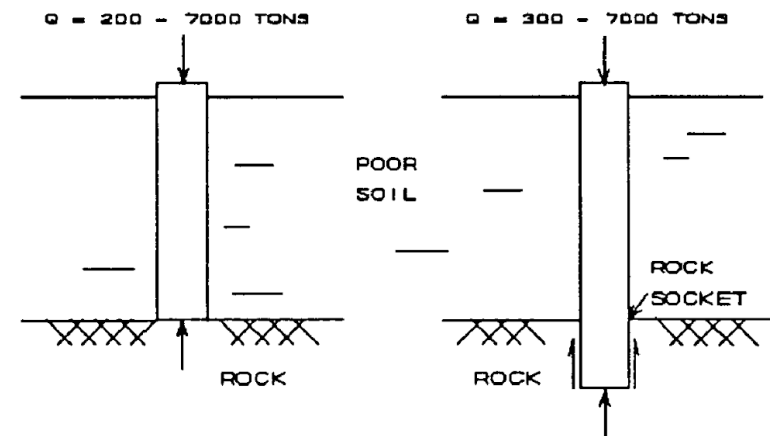
Load Transfer of Drilled Shafts



a. FLOATING SHAFTS IN HOMOGENEOUS SOIL

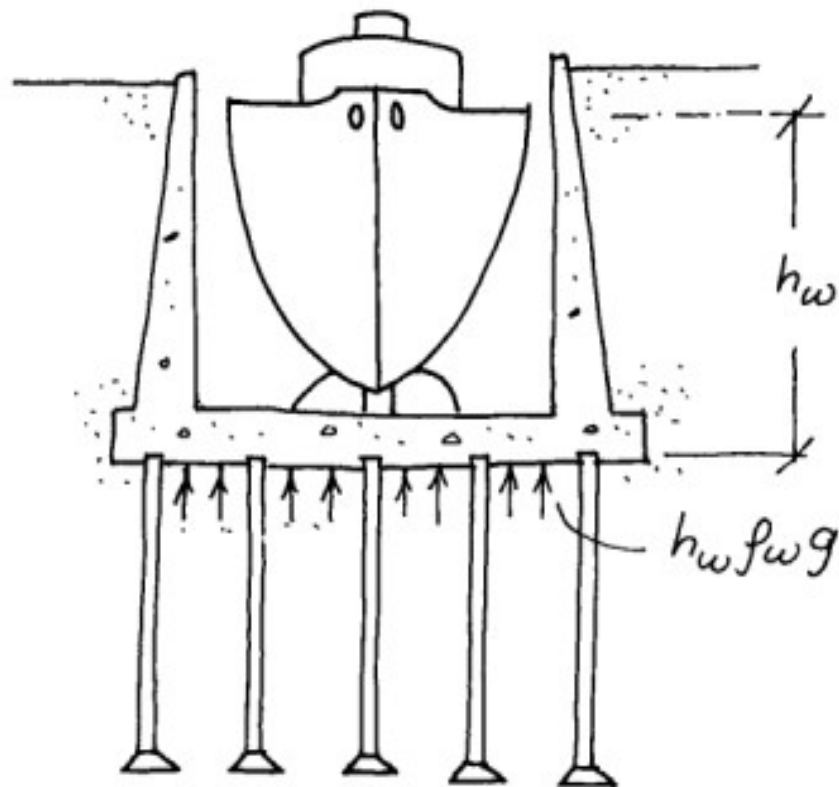


b. SHAFTS END BEARING IN HARD SOIL

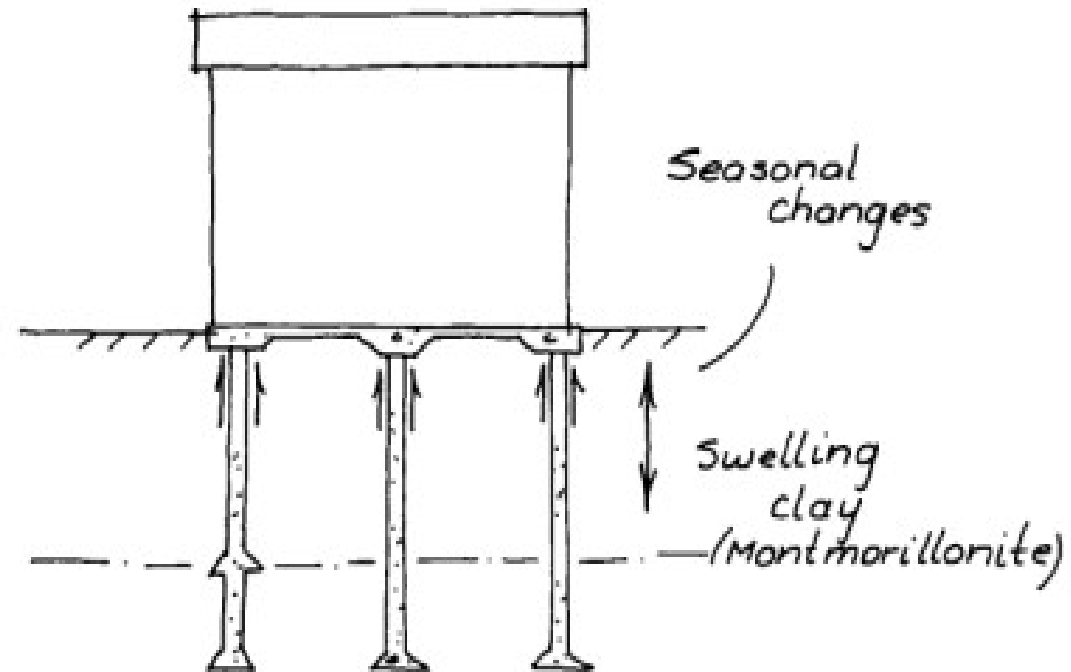


c. SHAFTS END BEARING IN ROCK

Applications of Drilled Shafts

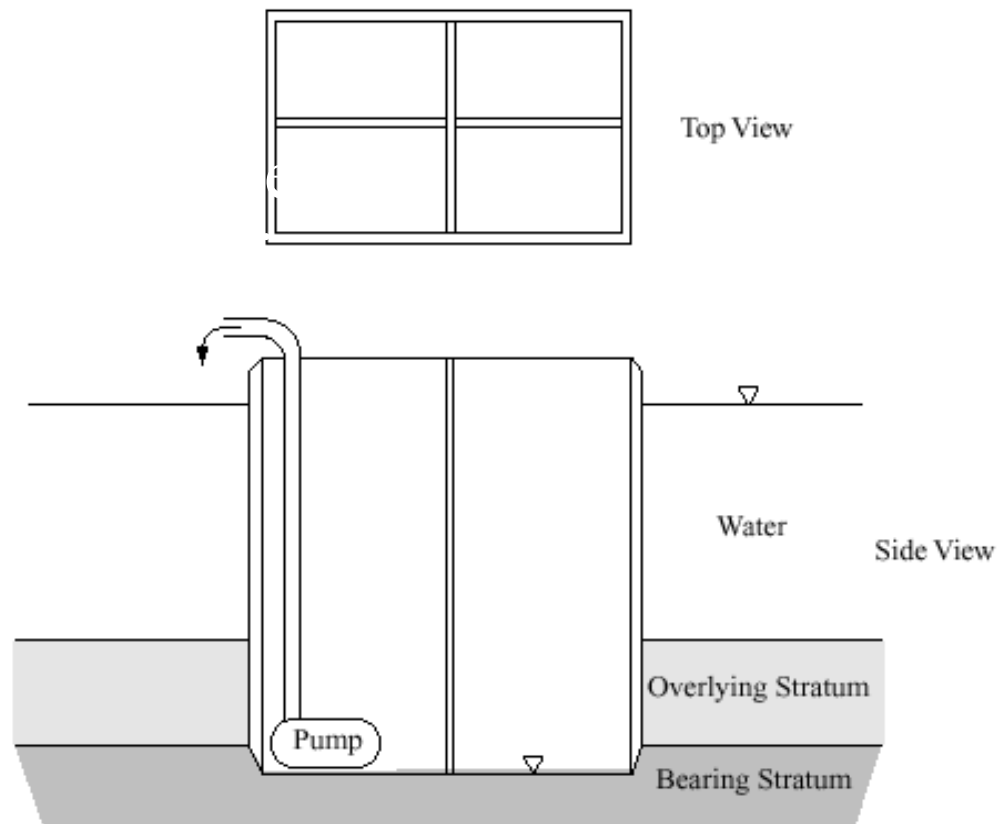


Dry docks
Subway stations



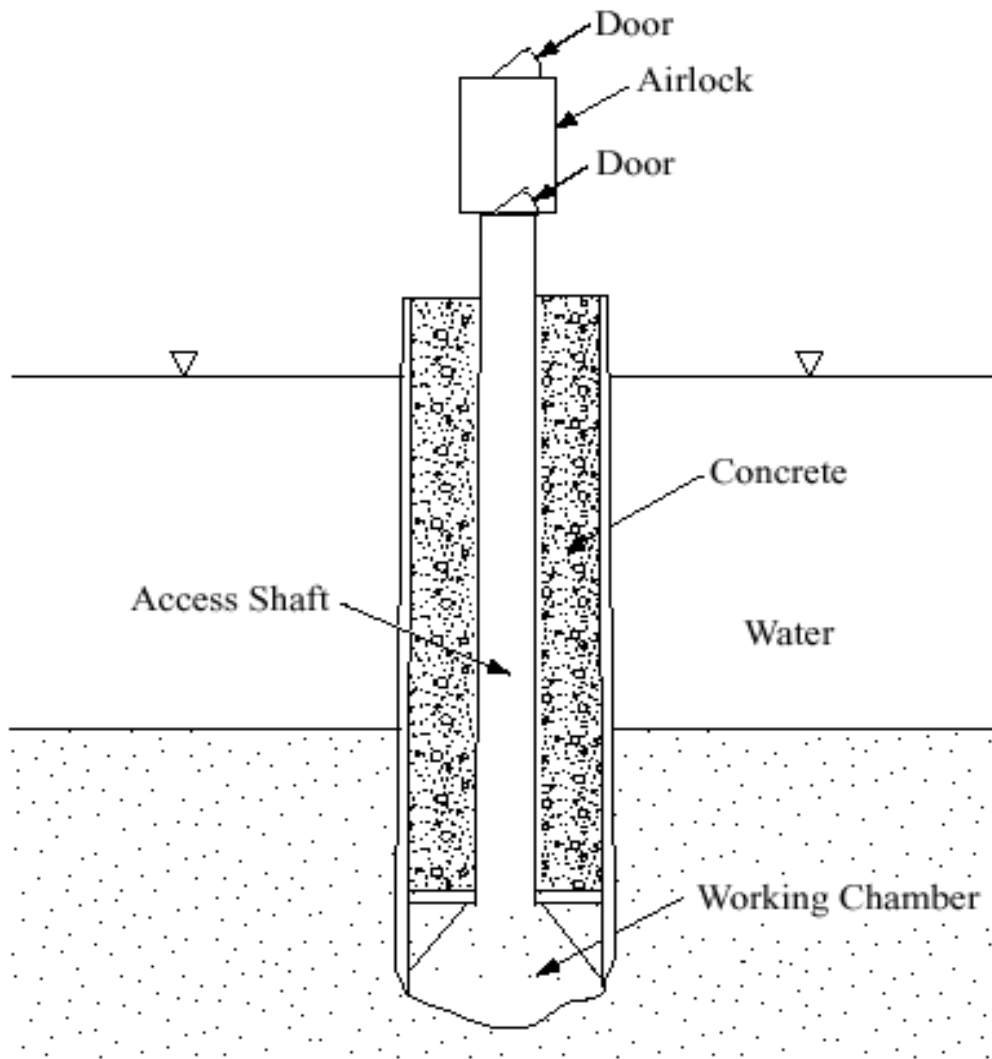
India: Black cotton soil
Texas: Beaumont clay

Caissons



- A prefabricated hollow box or cylinder which is sunk into the ground and then filled with concrete
- Are usually used in construction of bridge piers and other structure where the foundation is under water

Pneumatic Caissons



- Compressed air is used to keep water out and allow installation and construction in the dry
- High air pressures can and have created dangerous air conditions for workers, who must use an air lock

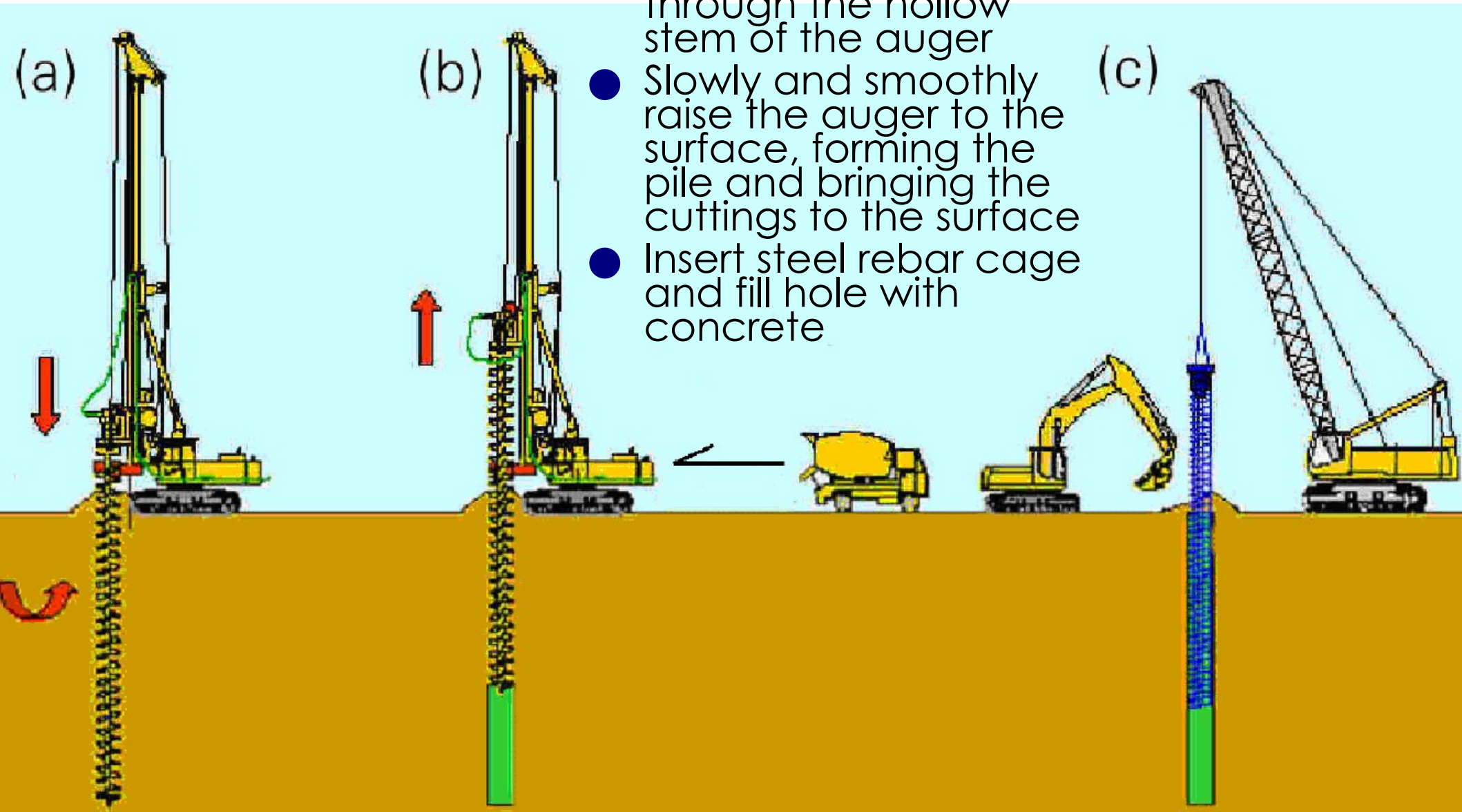
Auger-Cast Piles



- Other names
 - Augered Pressure Grouted (APG) Piles
 - Augered cast-in-place pile
 - Continuous flight auger pile
 - Intruded mortar piles
 - Augerpass Pile
 - Auger Pile
 - Grouted Bored Pile

Auger-Cast Piles

- Using a continuous flight auger 300-400 mm (12-16") in diameter, drill a hole in the ground
- Typical depths are 6 – 15 m (20'-50') but can go up to 27 m (90')
- Inject cement grout through the hollow stem of the auger
- Slowly and smoothly raise the auger to the surface, forming the pile and bringing the cuttings to the surface
- Insert steel rebar cage and fill hole with concrete



Advantages and Disadvantages of Auger-Cast Piles

● Advantages

- Low mobilisation cost
- Low noise and vibration
- Auger protects the hole from caving
- Grout is injected under pressure, so there is good soil bond and some soil compaction
- Used in a wide variety of soils

● Disadvantages

- Must have good contractor quality control and skills
- Auger can draw up more soil than it should under some conditions
- If equipment breaks down, pile is lost
- Cannot be used with cobbles and boulders
- No check on capacity

Static Geotechnical Capacity Analysis

Drilled Shafts

- Drilled Shafts do not compact the soils as driven piles do; they expand the soils, so soils can be looser around them, depending upon how long the hole remains open

● Analysis methods

- Empirical Methods
- Methods that deal with the additional uplift resistance of a belled toe (which we will deal with when we get to expansive soils)
- It is also possible to use t-z software to analyze drilled shafts, as is the case with driven piles

● Method of Analysis

- We use the O'Neill and Reese (1999) or "old" FHWA Drilled Shaft manual, but only because they use ASD rather than LRFD
- Soils for drilled shafts are classified as follows:
 - Cohesive Soils (cohesion < 5 ksf)
 - Granular Soils (SPT < 50)
 - Intermediate Geomaterials (IGM)
 - Cohesive, cohesion > 5 ksf
 - Granular, SPT > 50
- Rock, unconfined compressive strength > 100 ksf

Drilled Shafts in Clay Soils

- Shaft Capacity

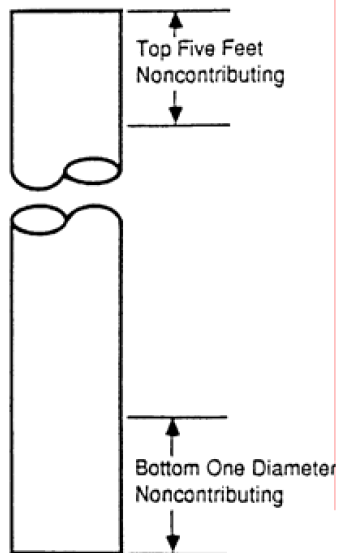
- Use alpha method, compute as follows:

$$\alpha = 0.55, c_u \leq 150 \text{ kPa or } 3 \text{ ksf}$$

$$\alpha = 0.45, c_u \geq 250 \text{ kPa or } 5 \text{ ksf}$$

Linearly interpolate for intermediate values

- Watch for excluded zones



$$f_s = \alpha c_u$$

- Toe Capacity

$$q'_b = N_c c_u$$

- q'_b = net unit toe-bearing pressure
- N_c = bearing capacity factor
 - 6.5 at $c_u < 25 \text{ kPa (500 psf)}$
 - 8.0 at $25 \text{ kPa (500 psf)} < c_u < 100 \text{ kPa (1000 psf)}$
 - 9.0 at $c_u > 100 \text{ kPa (2000 psf)}$
- c_u = undrained shear strength between the toe and $2B$ below the toe



Ultimate Capacity of Drilled Shafts in Sand

● Sand Shaft Friction

$$f_{si} = \beta_i \sigma'_{oi}$$

$$\beta_i = K_{si} \tan \delta_i$$

$$\beta_i = \frac{N_{60}}{15} \left[\frac{3}{2} - \sqrt{\frac{z_i}{16.66}} \right], N_{60} < 15 \text{ BPF (} z_i \text{ in meters)}$$

$$\beta_i = \frac{N_{60}}{15} \left[\frac{3}{2} - \sqrt{\frac{z_i}{54.87}} \right], N_{60} < 15 \text{ BPF (} z_i \text{ in feet)}$$

$$\beta_i = \left[\frac{3}{2} - \sqrt{\frac{z_i}{16.66}} \right], N_{60} \geq 15 \text{ BPF (} z_i \text{ in meters)}$$

$$\beta_i = \left[\frac{3}{2} - \sqrt{\frac{z_i}{54.87}} \right], N_{60} \geq 15 \text{ BPF (} z_i \text{ in feet)}$$

$$\beta_i = 2 - \left(\frac{z_i}{12.55} \right)^{0.75}, N_{60} \geq 15 \text{ BPF, Gravelly Sands or Gravels (} z_i \text{ in meters)}$$

$$\beta_i = 2 - \left(\frac{z_i}{41.16} \right)^{0.75}, N_{60} \geq 15 \text{ BPF, Gravelly Sands or Gravels (} z_i \text{ in feet)}$$

● Toe Resistance

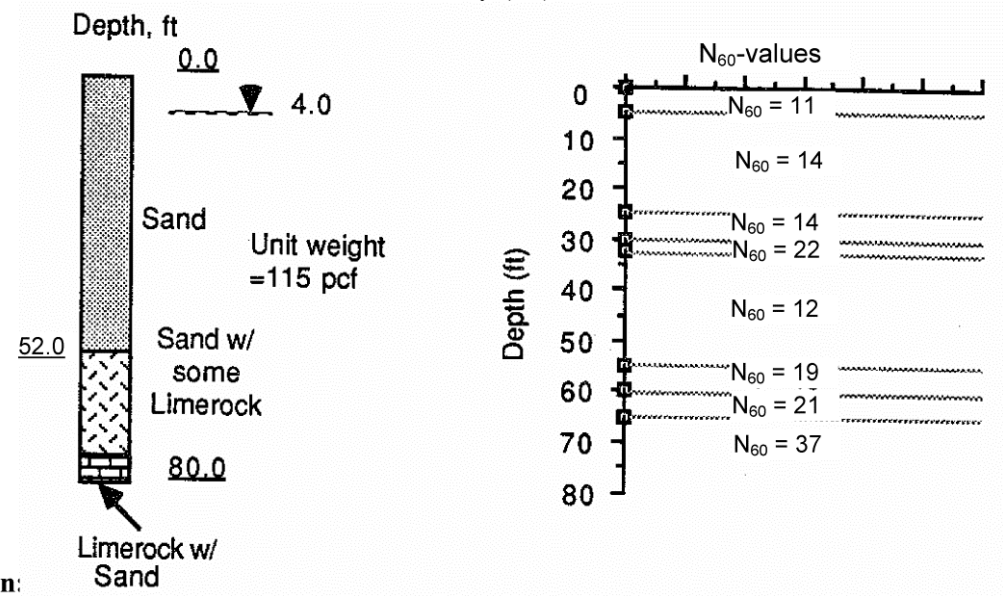
$$q_b = 1.2 N_{60} \leq 60 \text{ ksf (U.S.)}$$

$$q_b = 57.5 N_{60} \leq 2900 \text{ kPa (SI)}$$

- Method based on SPT results

Drilled Shaft Design Example

Example 9-5: Size a shaft to resist 170 tons of vertical design load in the soil profile shown below. Assume a factor of safety (FS) of 2.5.



Solution:

The ultimate geotechnical axial load = (FS) (Design Load) = (2.5) (170 tons) = 425 tons. Assume a straight-sided drilled shaft with a diameter of 3-ft and a length of 60-ft. Thus, $\pi(D) = 9.42$ -ft

Use Equation 9-41 to determine ultimate skin resistance, $Q_s = \pi D \sum_{i=1}^N \gamma_i z_i \beta_i \Delta z_i$

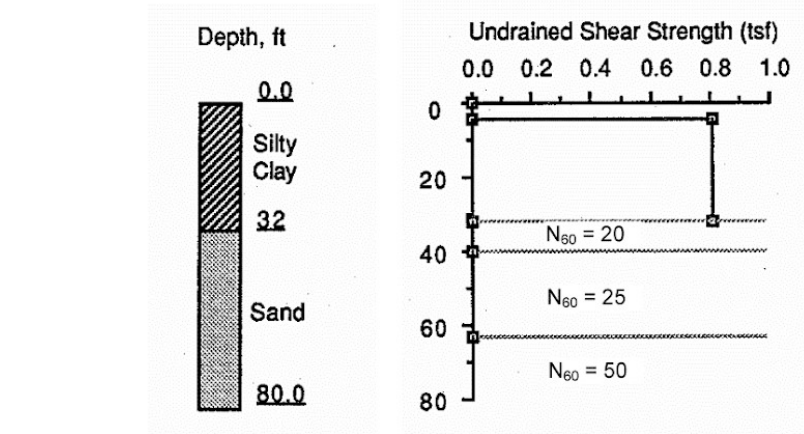
Depth Interval, Δz , ft	Surface Area per depth interval, $\Delta z(\pi)(D)$, ft ²	Average effective vertical (overburden) stress, $p_o = \gamma z_i$ tsf	β $\beta_i = 1.5 - 0.135\sqrt{z_i}$ with $1.2 > \beta_i > 0.25$	ΔQ_s Tons
0 – 4	37.7	0.115	1.20	5.20
4 – 30	245.0	0.572	0.94	131.70
30 – 60	282.7	1.308	0.59	218.20
Q_s				355.10

Base resistance ($N_{60}=21$ at 60-ft). Using Equation 9-45a $q_t = 1.2N_{60} = 25.2$ ksf = 12.6 tsf
 $A_t = 7.07$ ft² Therefore, $Q_t = (7.07 \text{ ft}^2) (12.6 \text{ tsf}) = 89.1$ tons

Thus, ultimate geotechnical axial resistance, Q_{ult} is given by:

$$Q_u = 355.1 + 89.1 = 444.2 \text{ tons} \approx 440 \text{ tons} > 425 \text{ tons} \quad \text{Okay.}$$

Example 9-6: Determine the shaft length to resist 150 tons of vertical design load in the mixed (clay on sand) soil profile shown below. Assume a safety factor of 2.5. Assume a total unit weight of 125 pcf for clay and 115 pcf for sand. Water table is at a depth of 17-ft. Assume depth of zone of seasonal moisture change to be 5-ft. Once the shaft is sized for ultimate load, check the deformation under design load of 150 tons.



Solution:

For a factor of safety of 2.5, the ultimate axial load is computed to be (2.5)(150 tons) = 375 tons.

For a straight-sided shaft with a diameter of 3.0-ft and a depth of penetration of 50-ft, $\pi(D) = 9.42\text{-ft}$

Use Equation 9-36 and 9-41,

$$Q_s = \pi D \sum_{i=1}^N \alpha_i s_{ui} \Delta z_i$$

$$Q_s = \pi D \sum_{i=1}^N \gamma_i z_i \beta_i \Delta z_i$$

Soil	Depth Interval, Δz , ft	Surface Area per depth interval, $\Delta z(\pi)(D)$, ft ²	Shear Strength or Average effective vertical (overburden) stress, tsf	α or β	ΔQ_s , Tons
Clay	0 – 5	--	--	0.00	0
Clay	5-32	254.5	0.80 (shear strength)	$\alpha = 0.55^*$	112.0
Sand	32-50	169.6	$\{(17 \text{ ft} \times 125 \text{ pcf}) + (32 \text{ ft} - 17 \text{ ft})(125 \text{ pcf} - 62.4 \text{ pcf}) + 9 \text{ ft}(115 \text{ pcf} - 62.4 \text{ pcf})\} / 2,000 = 3537.4 \text{ psf} / 2,000 = 1.769 \text{ tsf}$	$\beta = 0.64^{**}$	192.0
* From Equation 9-37a ** From Equation 9-42, $\beta_i = 1.5 - 0.135\sqrt{z_i}$ At mid-depth of sand layer, $z_i = 32 \text{ ft} + (50 \text{ ft} - 32 \text{ ft}) / 2 = 41 \text{ ft}$ At $z_i = 41 \text{ ft}$, $\beta_i = 1.5 - 0.135\sqrt{41 \text{ ft}} \approx 0.64$					Qs 304.0

Base resistance ($N_{60}=25$ at 50 ft)

Use Equation 9-45a

$$q_t = 1.2 N_{60} = 1.2 (25) = 30 \text{ ksf} = 15 \text{ tsf}$$

$$A_t = 7.07 \text{ ft}^2$$

$$Q_t = (7.07 \text{ ft}^2) (15.0 \text{ tsf}) = 106 \text{ tons}$$

Total ultimate axial resistance, Q_{ult} is given by:

$$Q_u = 304.0 + 106.0 = 416.0 \text{ tons} > 375 \text{ tons} \quad \text{Okay.}$$

Check of settlement under design load (150 tons)

Because most of the load in side resistance and all of the end bearing are derived from sand, Figures 9-52 and 9-53 will be used to estimate settlement. A settlement near the upper bound in both figures will be selected as a conservative estimate.

A settlement of 0.15 percent of the diameter is selected for the average settlement of the sides, or 0.06-inch. That would indicate that about 138 tons is carried in side resistance, and about 12 tons is carried in bearing, assuming that the shaft is essentially incompressible.

Comment: The settlement solution appears to be reasonable.

Drilled Shaft Design Example

Questions?

