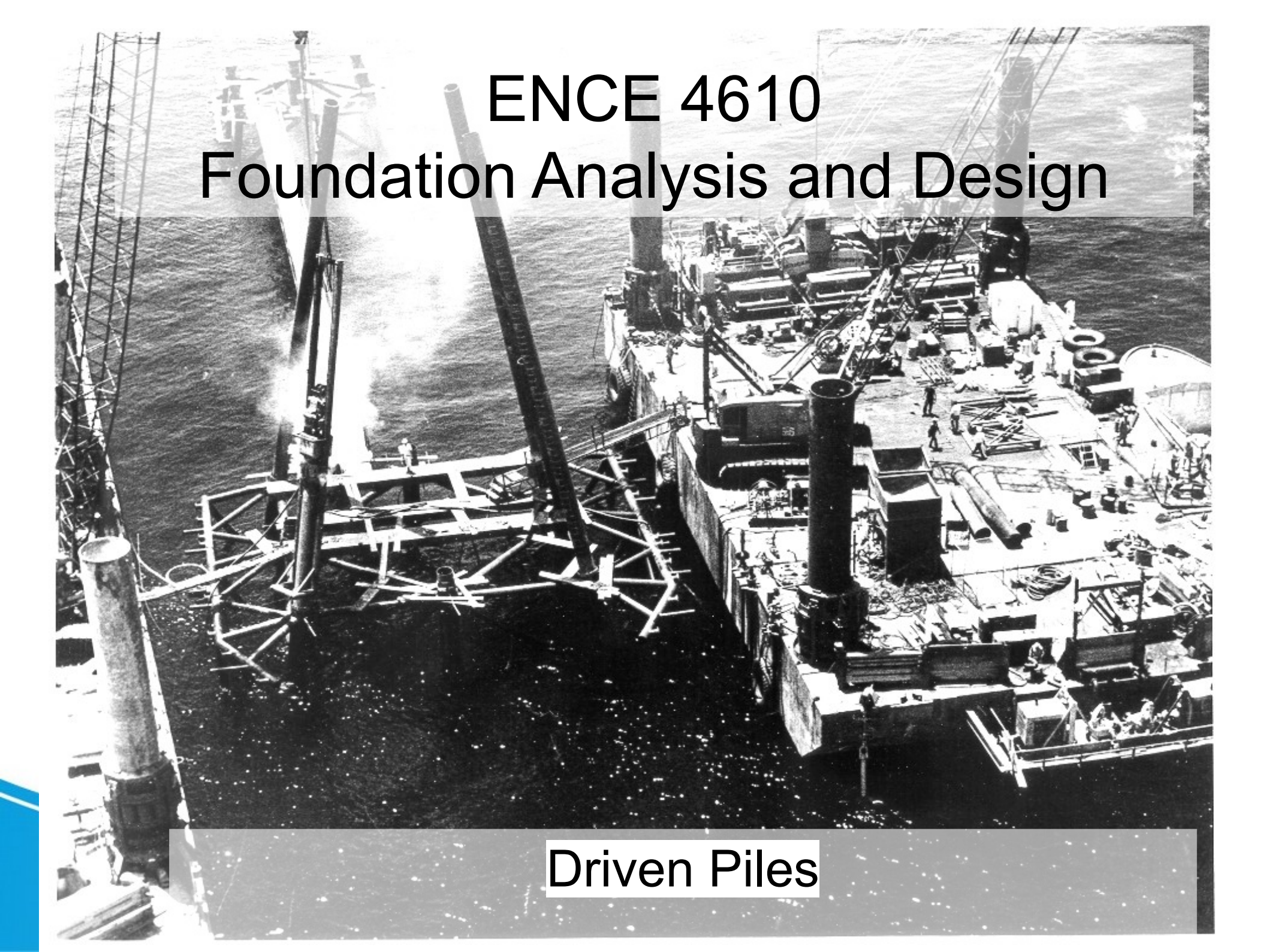


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ENCE 4610

Foundation Analysis and Design

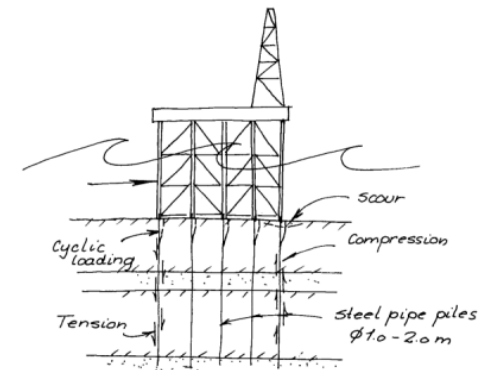
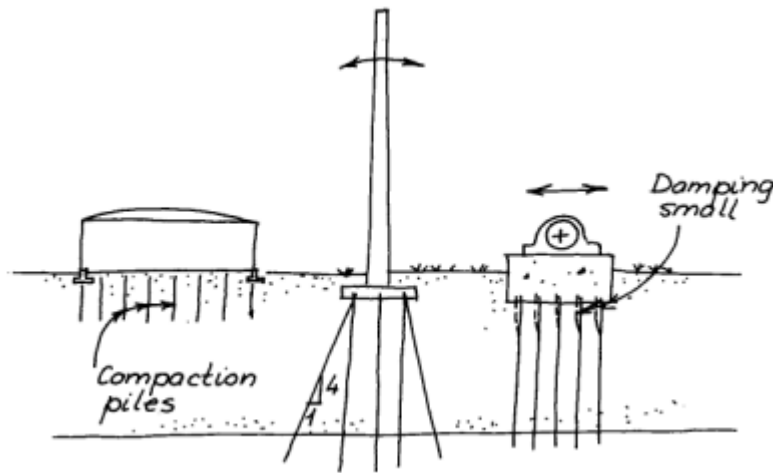
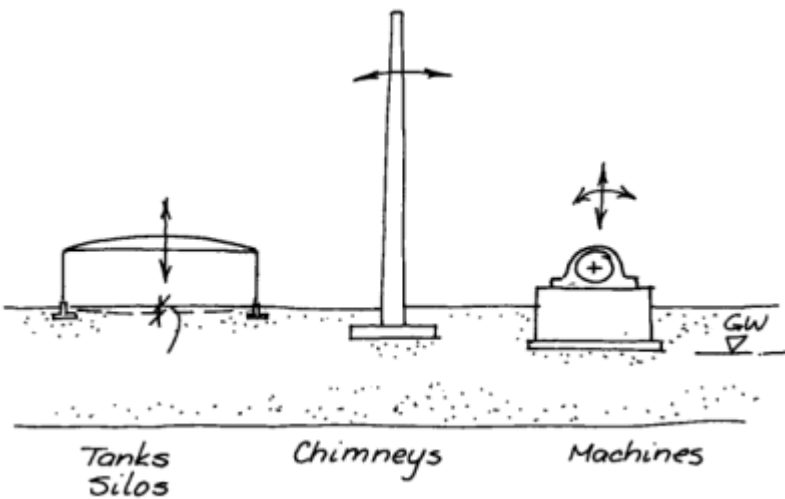
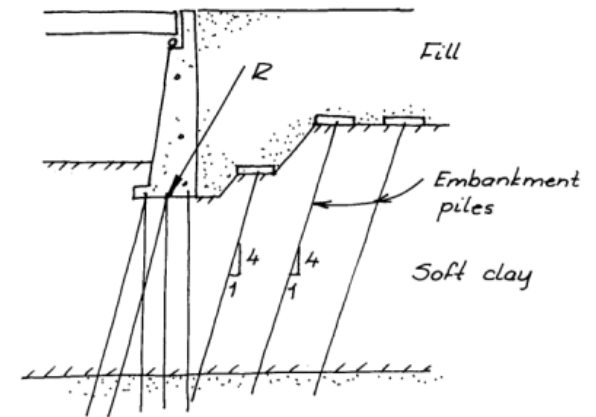
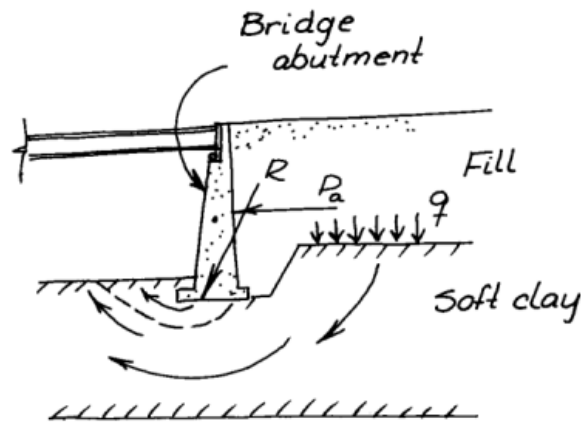
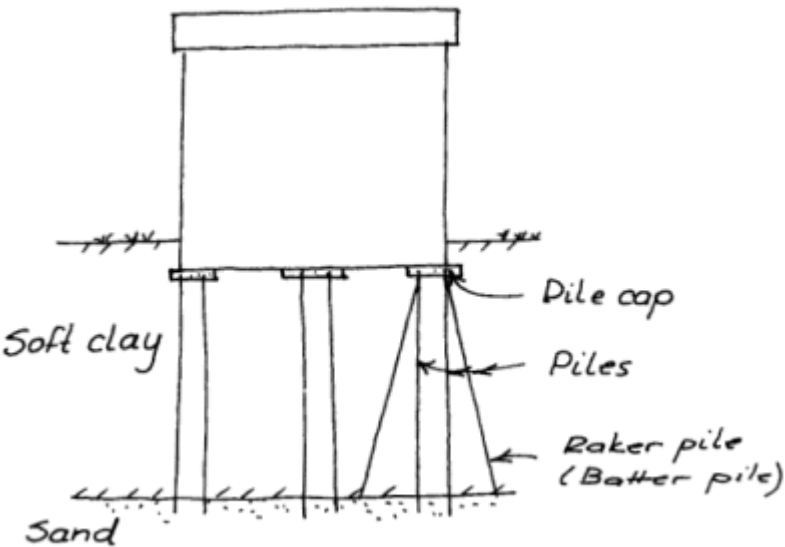
Driven Piles

Why Deep Foundations?

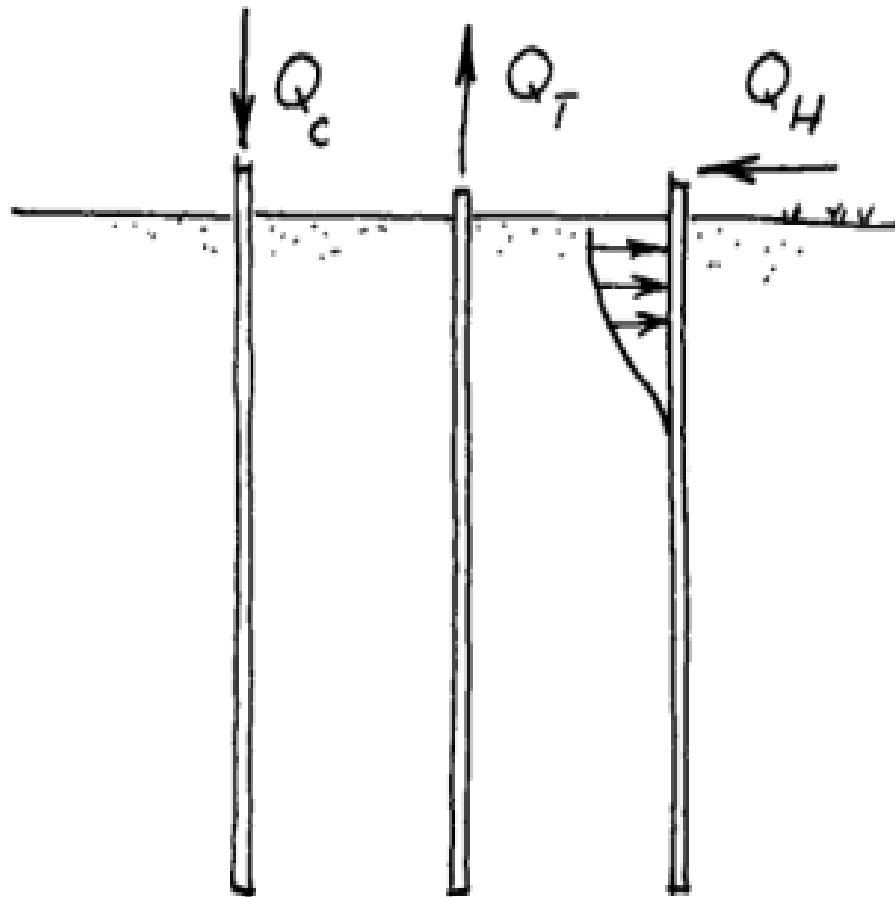
“Foundation design and construction involves assessment of factors related to engineering and economics. As discussed in Chapter 8, the selection of the most feasible foundation system requires consideration of both shallow and deep foundation types in relation to the characteristics and constraints of the project and site conditions. Situations commonly exist where shallow foundations are inappropriate for support of structural elements. These situations may be related either to the presence of unsuitable soil layers in the subsurface profile, adverse hydraulic conditions, or intolerable movements of the structure. Deep foundations are designed to transfer load through unsuitable subsurface layers to suitable bearing strata.”



Reasons for Deep Foundations



Loading of Deep Foundations



(a) Compression

(b) Tension

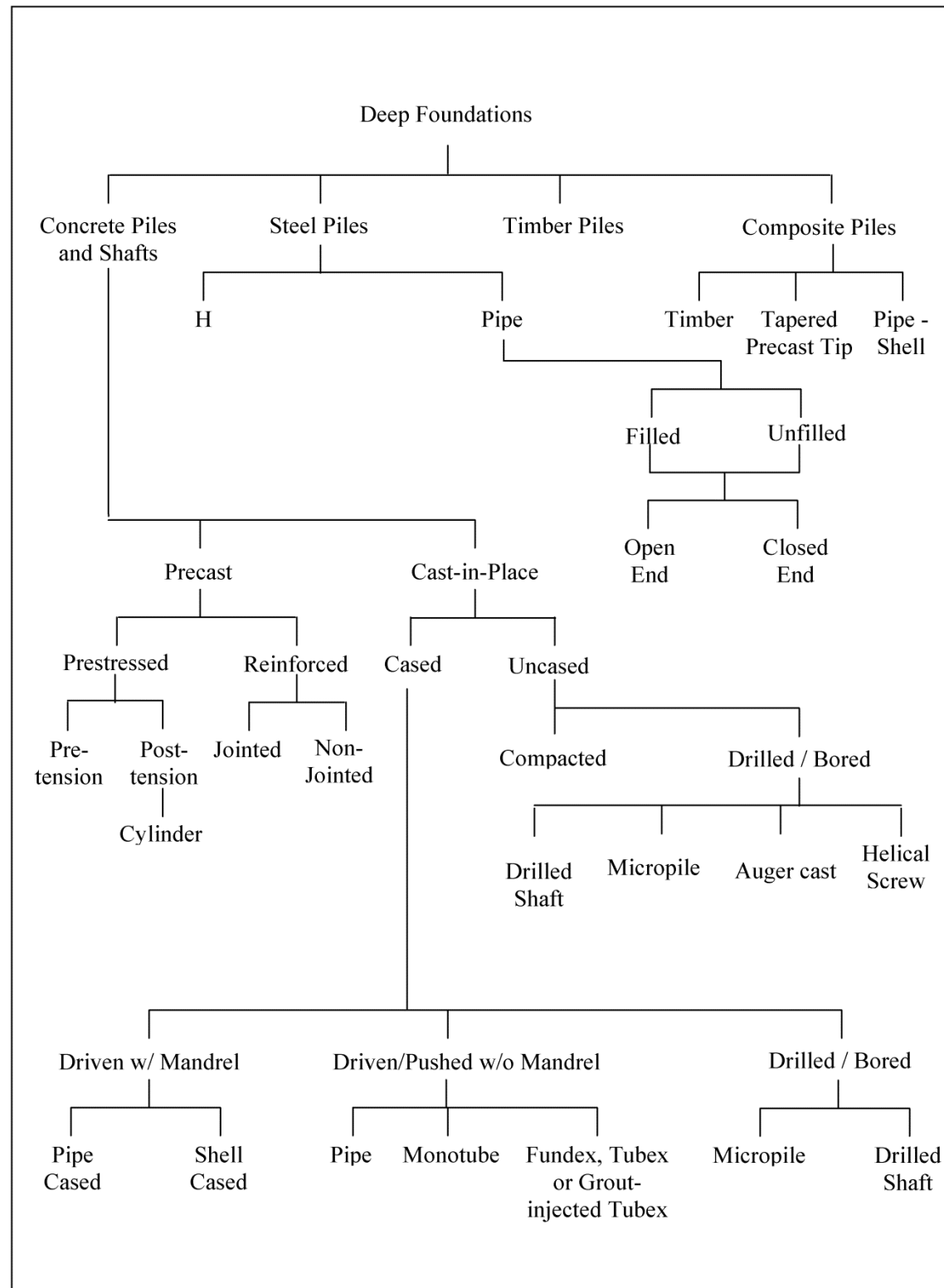
(c) Lateral load

- 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

● We will consider two main types of deep foundations

- Driven Piles
- Drilled Shafts (Auger Cast piles have a similar design process, but different installation)



Outline of Driven Piles

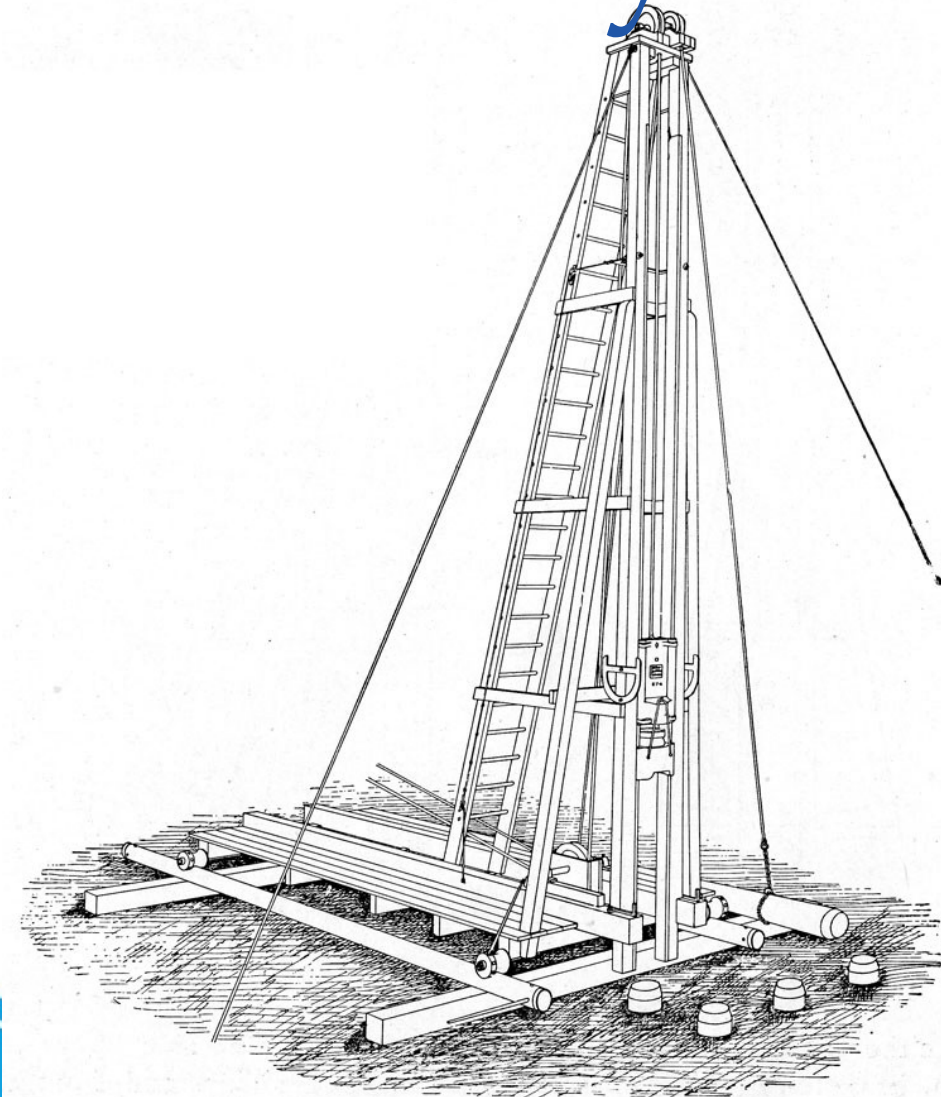
- Rationale for Deep Foundations in General
- Overview of Driven Piles and Pile Driving
- Design Process of Driven Piles and Pile Driving Equipment
- Factor of Safety Considerations
- Single Pile Static Design
 - Fellenius β Method
 - Soil Set-up
 - TAMWAVE Software
 - Settlement Calculations
- Design of Pile Groups
 - Bearing Capacity and Settlement
- Downdrag
- Topics to be Covered Later
 - Static Load Testing of Piles
 - Dynamic Load Testing of Piles

Driven Piles

A long, slender, prefabricated structural member driven by impact or vibration, or otherwise inserted into the ground



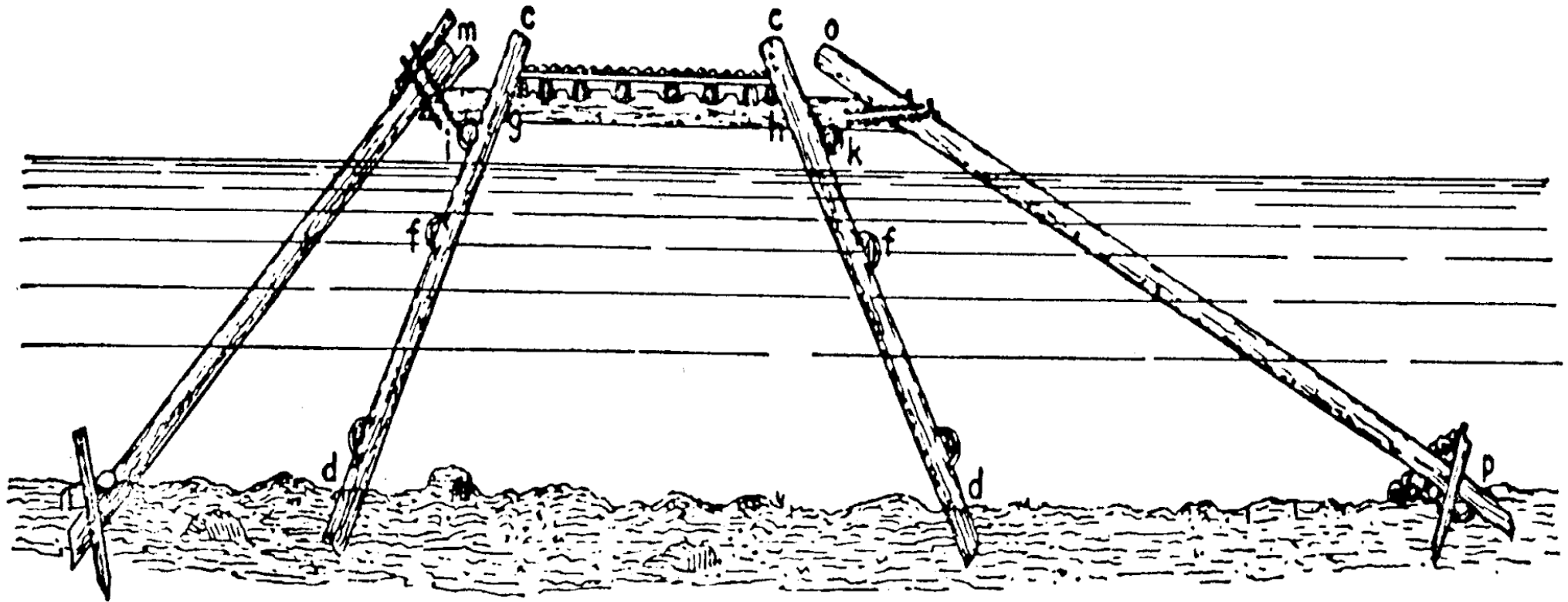
History of Driven Piles



- The oldest form of deep foundation in existence
- All ancient civilizations (Greek, Roman, Chinese) used driven piles to support structures in poor soils
- Driving equipment involved positioning, raising and lowering the driver by hand

Caesar's Bridge over the Rhine

- Built in a span of ten days
- Consisted of batter piles for the lateral loads of the river



Advances in Driven Piles and Pile Drivers

- Hammers

- Steam Hammers

- Naysmith (1845) – related to forging hammers
 - Vulcan (1887), MKT, etc.

- Pile Driving Rigs

- Rotating Skid Rigs
 - Crawler mounted rigs

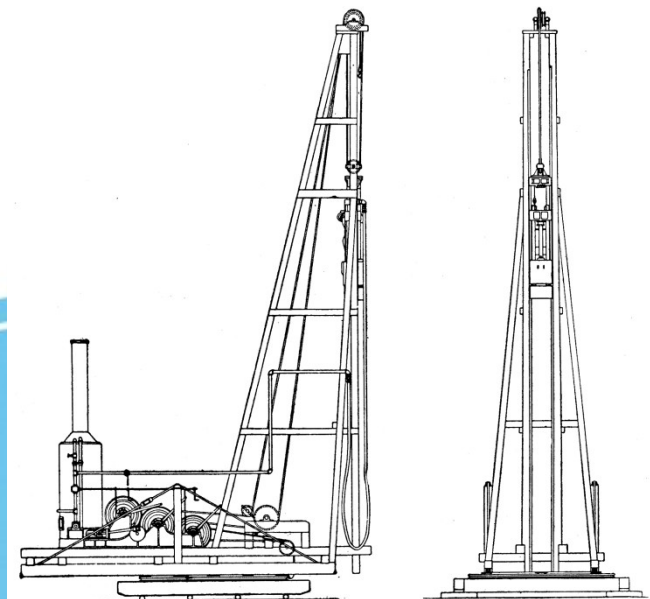
- Piles

- Steel Piles

- H-Piles – addressed problems of bridge scour in the Midwest
 - Pipe Piles

- Precast Concrete Piles

- François Hennebique (1897) – first use
 - A.A. Raymond (1901) – built with it Raymond Concrete Pile Company



Types of Piles

● Types of Piles Used

- Timber Piles
- Steel Piles
 - H-Beams
 - Pipe Piles
- Concrete Piles
 - Precast Piles
 - Prestressed Piles
- Other Types of Piles
 - Plastic-Steel Composites
 - Sheet Piling



● Selection of Pile Type

- Applied Loads
- Required Diameter
- Required Length
- Local availability of each pile type
- Durability of the pile material in a specific environment
- Anticipated driving conditions

Timber Piles

- The oldest type of pile in use
- Most used today are Southern Pine or Douglas Fir

Table 6-1 Timber Piles Technical Summary

PILE TYPE	TIMBER PILES
TYPICAL LENGTHS	15 to 75 feet for Southern Pine. 15 to 120 feet for Douglas Fir.
MATERIAL SPECIFICATIONS	ASTM D25. AWPA UC4A, UC4B, UC4C, UC5A, UC5B and UC5C.
TYPICAL FACTORED RESISTANCE	50 to 120 kips.
MAXIMUM DRIVING STRESS	$\sigma_{dr} = \phi_{da} (F_{co})$. $\phi_{da} = 1.15$. $F_{co} = 1.25$ ksi for Douglas Fir, 1.20 ksi for Southern Pine.
ADVANTAGES	• Comparatively low in initial cost. • Permanently submerged piles are resistant to decay. • Easy to handle.
DISADVANTAGES	• Difficult to splice. • Vulnerable to damage in hard driving; both pile head and pile toe may need protection. • Intermittently submerged piles are vulnerable to decay unless treated.
REMARKS	• Best suited for friction piles in granular material. • Suitable for friction piles with lower factored resistances in cohesive soils.

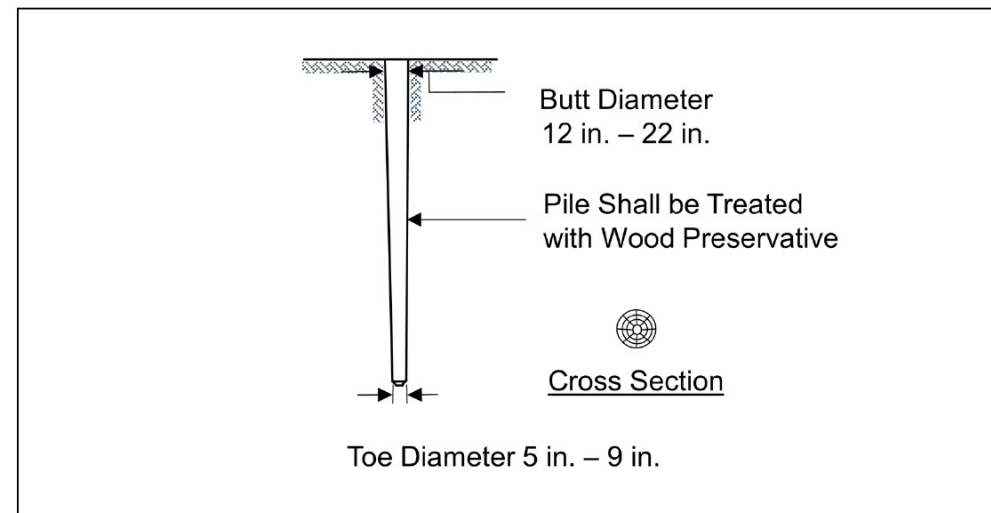


Figure 6-2 Timber pile typical illustration.

Steel Piles

- Steel piles first came into use in the 1890's
- Can be driven through hard soils and carry large loads
- Especially useful for tension and uplift loads
- Easy to splice and add onto
- Expensive compared to other piles
- May be subject to corrosion, depending upon soil and water conditions



H-Piles

- Similar to WF beams used in structural applications but flanges and webs are of equal thickness with H-beams
- Often used as end bearing piles in rock



Table 6-2 Steel H-Piles Technical Summary

PILE TYPE	STEEL – H-PILES
TYPICAL LENGTHS	15 to 200 feet.
MATERIAL SPECIFICATIONS	ASTM - A572, A588, or A690 Grade 50, 60. (A572 Grade 50 is standard).
TYPICAL FACTORED RESISTANCE	260 to 1,600 kips.
MAXIMUM DRIVING STRESS	$\sigma_{dr} = 0.9 \phi_{da} F_y$ $\phi_{da} = 1.00$ F_y = Yield strength of steel (ksi).
ADVANTAGES	• Available in various lengths and sizes. • High factored resistance. • Small soil displacement. • Easy to splice. • Pile toe protection will assist penetration through harder layers and some small obstructions.
DISADVANTAGES	• Vulnerable to corrosion where exposed and in corrosive soil conditions. • HP section may be damaged or deflected by major obstructions.
REMARKS	• Best suited for toe bearing on rock. • Factored resistance reduced in corrosive environments. • Length and cost overruns often occur when used as a friction pile in granular materials.

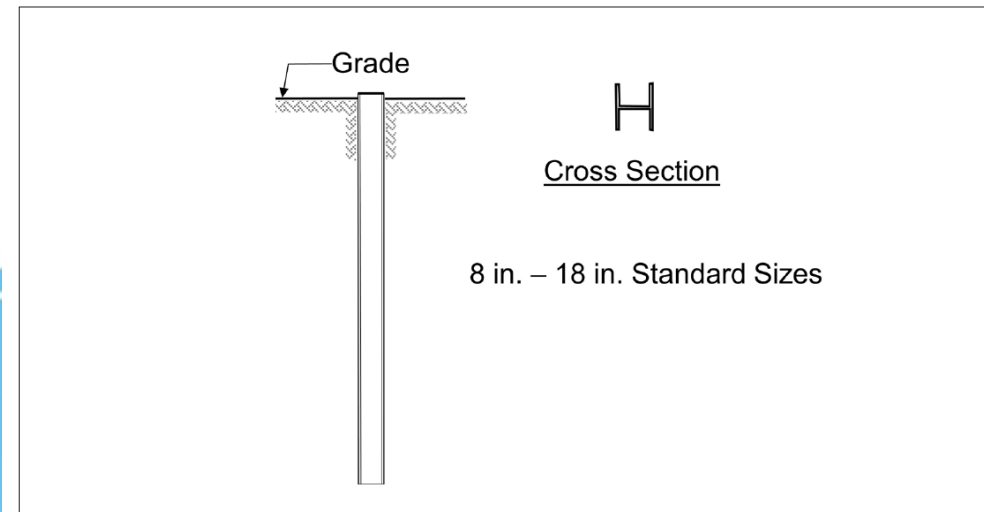
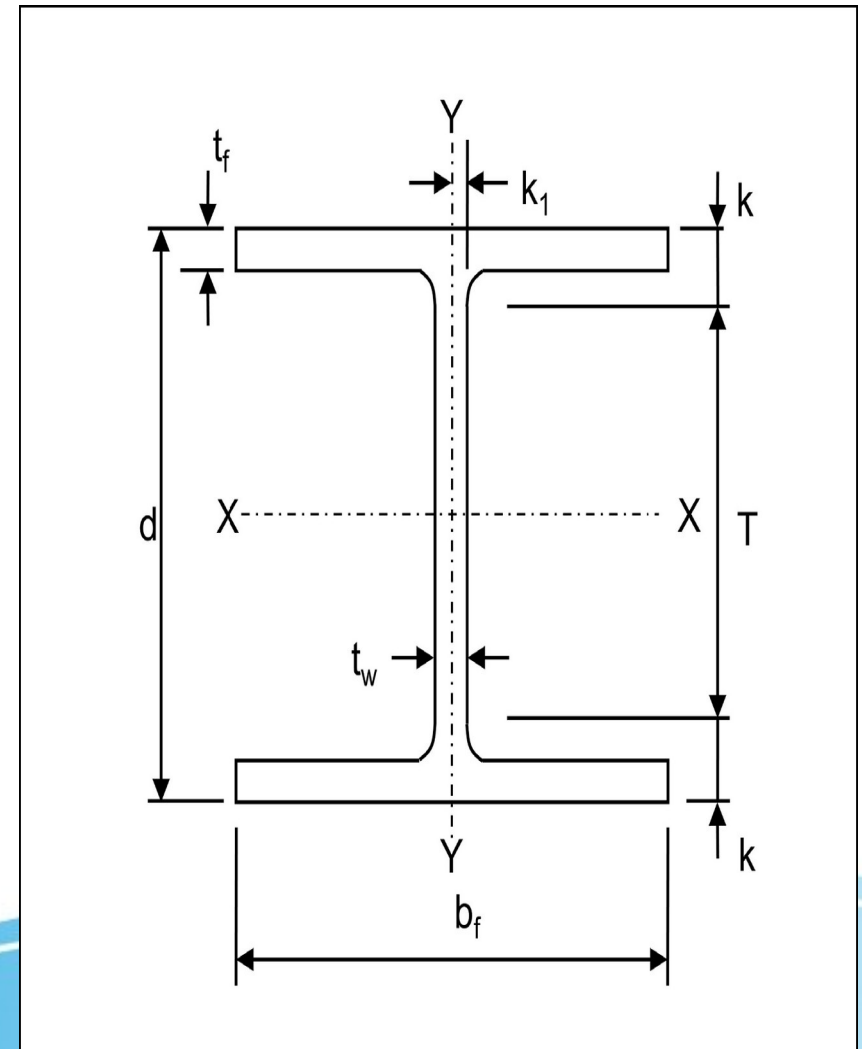


Figure 6-3 Steel H-pile typical illustration.

H-Pile Chart

Section	Weight lb/ft	Area in ²	Depth d in	Flange Width b _f in	Thickness		Properties							
					Web t _w in	Flange t _f in	X- Axis				Y-Axis			
							I in ⁴	Z in ³	S in ³	r in	I in ⁴	Z in ³	S in ³	r in
HP8X36	36	10.6	8.02	8.16	0.445	0.445	119	33.6	29.8	3.36	40.3	15.2	9.9	1.95
HP10X42	42	12.4	9.70	10.10	0.420	0.415	210	48.3	43.4	4.13	71.7	21.8	14.2	2.41
HP10X57	57	16.7	9.99	10.20	0.565	0.565	294	66.5	58.8	4.18	101	30.3	19.7	2.45
HP12X53	53	15.5	11.80	12.00	0.435	0.435	393	74.0	66.7	5.03	127	32.2	21.1	2.86
HP12X63	63	18.4	11.90	12.10	0.515	0.515	472	88.3	79.1	5.06	153	38.7	25.3	2.88
HP12X74	74	21.8	12.10	12.20	0.610	0.605	569	105.0	93.8	5.11	186	46.6	30.4	2.92
HP12X84	84	24.6	12.30	12.30	0.685	0.685	650	120.0	106.0	5.14	213	53.2	34.6	2.94
HP12x89	89	25.9	12.36	12.32	0.720	0.720	689	126.3	111.6	5.16	225	56.2	36.5	2.94
HP12X102	102	29.9	12.56	12.64	0.819	0.819	811	147.6	129.3	5.20	276	67.1	43.7	3.04
HP12X117	117	34.4	12.76	12.87	0.929	0.929	946	170.8	148.2	5.24	331	79.3	51.4	3.11
HP14X73	73	21.4	13.60	14.60	0.505	0.505	729	118	107	5.84	261	54.6	35.8	3.49
HP14X89	89	26.1	13.80	14.70	0.615	0.615	904	146	131	5.88	326	67.7	44.3	3.53
HP14X102	102	30.1	14.00	14.80	0.705	0.705	1050	169	150	5.92	380	78.8	51.4	3.56
HP14X117	117	34.4	14.20	14.90	0.805	0.805	1220	194	172	5.96	443	91.4	59.5	3.59
HP16X88	88	25.8	15.30	15.70	0.540	0.540	1110	161	145	6.56	349	68.2	44.5	3.68
HP16X101	101	29.9	15.50	15.80	0.625	0.625	1300	187	168	6.59	412	80.1	52.2	3.71
HP16X121	121	35.8	15.80	15.90	0.750	0.750	1590	226	201	6.66	504	97.6	63.4	3.75
HP16X141	141	41.7	16.00	16.00	0.875	0.875	1870	264	234	6.70	599	116.0	74.9	3.79
HP16X162	162	47.7	16.30	16.10	1.000	1.000	2190	306	269	6.78	697	134.0	86.6	3.82
HP16X183	183	54.1	16.50	16.30	1.130	1.130	2510	349	304	6.81	818	156.0	100.0	3.89
HP18X135	135	39.9	17.50	17.80	0.750	0.750	2200	281	251	7.43	706	122.0	79.3	4.21
HP18X157	157	46.2	17.70	17.90	0.870	0.870	2570	327	290	7.46	833	143.0	93.1	4.25
HP18X181	181	53.2	18.00	18.00	1.000	1.000	3020	379	336	7.53	974	167.0	108.0	4.28
HP18X204	204	60.2	18.30	18.10	1.130	1.130	3480	433	380	7.60	1120	191.0	124.0	4.31



From Skyline Steel

Pipe Piles



Table 6-3 Steel Pipe Piles Technical Summary

PILE TYPE	STEEL – PIPE PILES
TYPICAL LENGTHS	15 to 200 feet.
MATERIAL SPECIFICATIONS	ASTM A252 Grade 2 or 3, API 5L, or API 2B - for pipe. ACI 318 - for concrete (if filled). ASTM A572 - for core (if used).
TYPICAL FACTORED RESISTANCE	100 to 1,250 kips (closed end, $D \leq 30$ in.) with concrete fill. 660 to 6,500 kips (open end, 16 in. $\leq D \leq 72$ in.) no concrete.
MAXIMUM DRIVING STRESS	$\sigma_{dr} = 0.9 \phi_{da} F_y$. $\phi_{da} = 1.00$ for non-composite during driving. F_y = Yield strength of steel (ksi).
ADVANTAGES	• Closed end pipe can be internally inspected after driving. • Low soil displacement for open end installation. • High factored resistances depending on section. • Open end pipe with shoe can be used for obstructions. • Open end pipe can be cleaned out and driven further. • Easy to splice.
DISADVANTAGES	• Vulnerable to corrosion where exposed and in corrosive soil conditions. • Potential soil displacement from larger closed end pipe.
REMARKS	• Provides high bending resistance where unsupported length is loaded laterally. • Open end not recommended as a friction pile in granular material due to tendency for length and cost overruns.

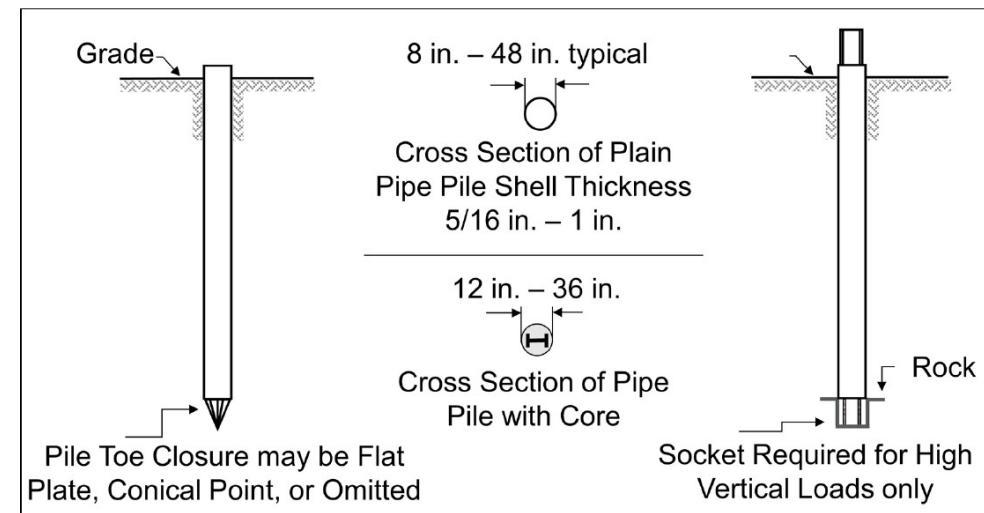


Figure 6-4 Steel pipe piles typical illustration.

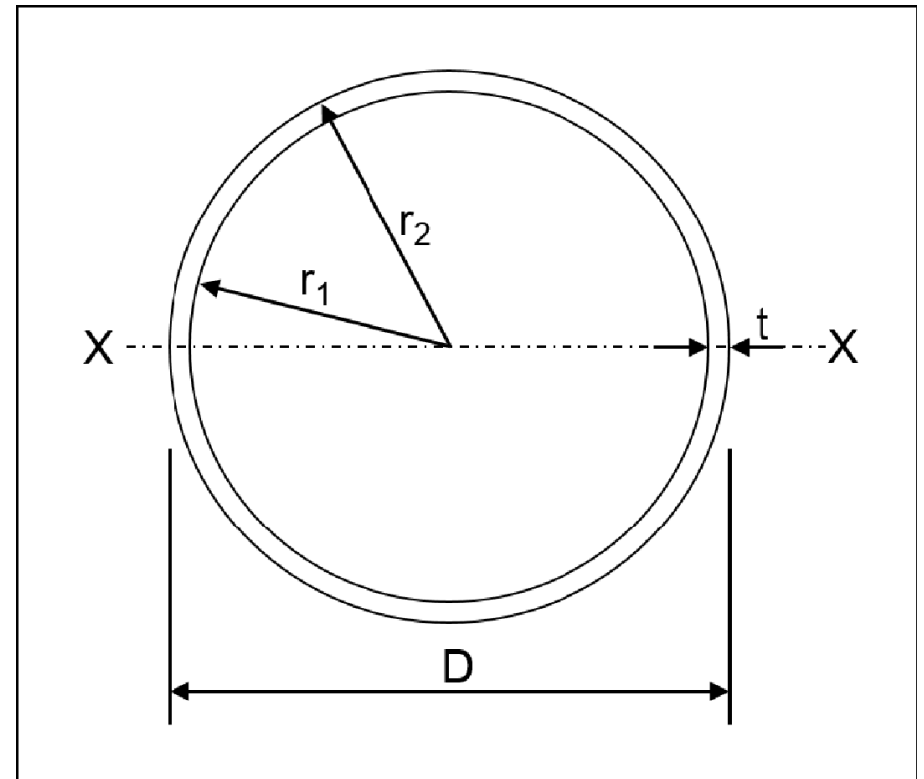
● Pipe Piles and Steel Pile Materials

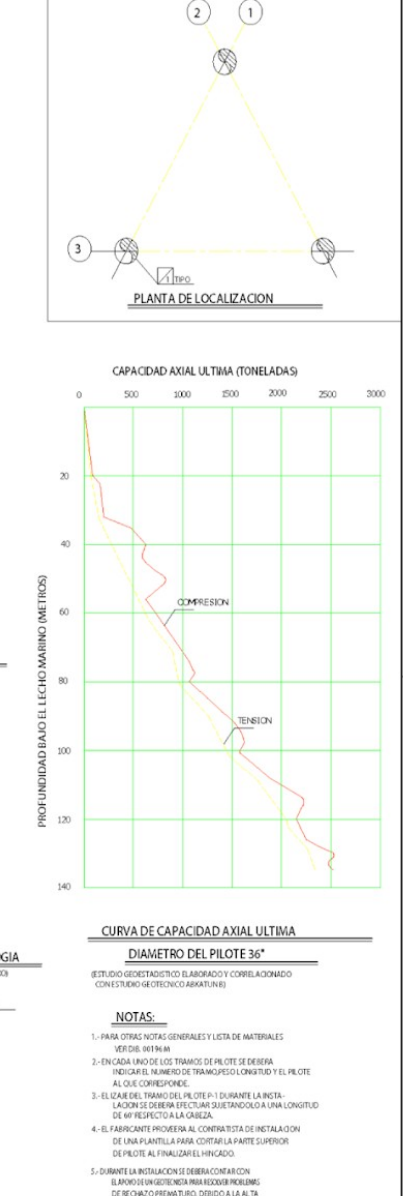
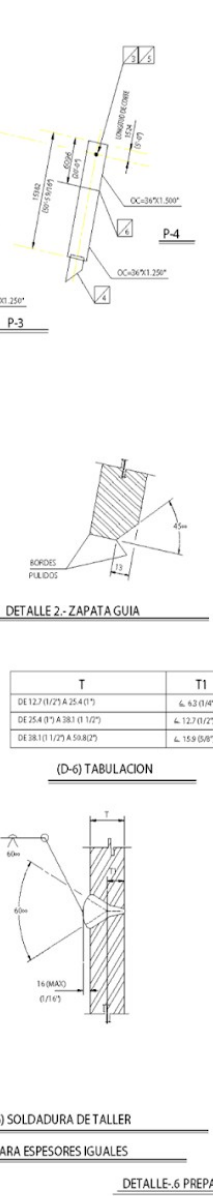
Table 8-1 Common Steel Pipe Pile Grades and Yield Stress

Designation/Grade	Yield Stress, F_y , ksi
ASTM A-252 Grade 2	35
ASTM A-252 Grade 3	45
ASTM A-252 Grade 3 (Mod)	50-80

Table 8-2 Common Steel H-pile Grades and Yield Stress

Designation/Grade	Yield Stress, F_y , ksi
A-36	36
ASTM A-572-50	50
ASTM A-572-60	60





Concrete Piles

- Either precast or prestressed piles
- Precast contain rebar similar to shallow foundations – not commonly used in North America
- Prestressed uses cables tensioned before concrete is poured



Square Concrete Piles



Concrete Cylinder Piles



Technical Overview of Concrete Piles

Table 6-4 Precast, Prestressed Concrete Technical Summary

PILE TYPE	PRECAST PRESTRESSED CONCRETE PILES
TYPICAL LENGTHS	30 to 150 feet.
MATERIAL SPECIFICATIONS	ACI 318 - for concrete. ASTM - A82, A615, A722, and A884 - for reinforcing steel. ASTM - A416, A421, and A882 - for prestressing.
TYPICAL FACTORED RESISTANCE	350 to 2,200 kips on solid square piles. 1,500 to 3,000 kips on spun cast cylinder piles.
MAXIMUM DRIVING STRESS	$\sigma_{dr} = \phi_{da} (0.85 f'_c - f_{pe})$ In compression. $\sigma_{dr} = \phi_{da} (0.095 \sqrt{f'_c} + f_{pe})$ In tension (normal conditions). $\sigma_{dr} = \phi_{da} (f_{pe})$ In tension (severe conditions). $\phi_{da} = 1.00$. f'_c = Concrete compressive strength (ksi). f_{pe} = Effective prestress (ksi).
ADVANTAGES	• High factored resistances. • Corrosion resistance obtainable. • Hard driving possible.
DISADVANTAGES	• Vulnerable to handling damage. • Can have relatively high breakage rate. • Potential soil displacement effects from large sections. • Difficult to splice when insufficient length ordered.
REMARKS	• Cylinder piles are well suited for bending resistance.

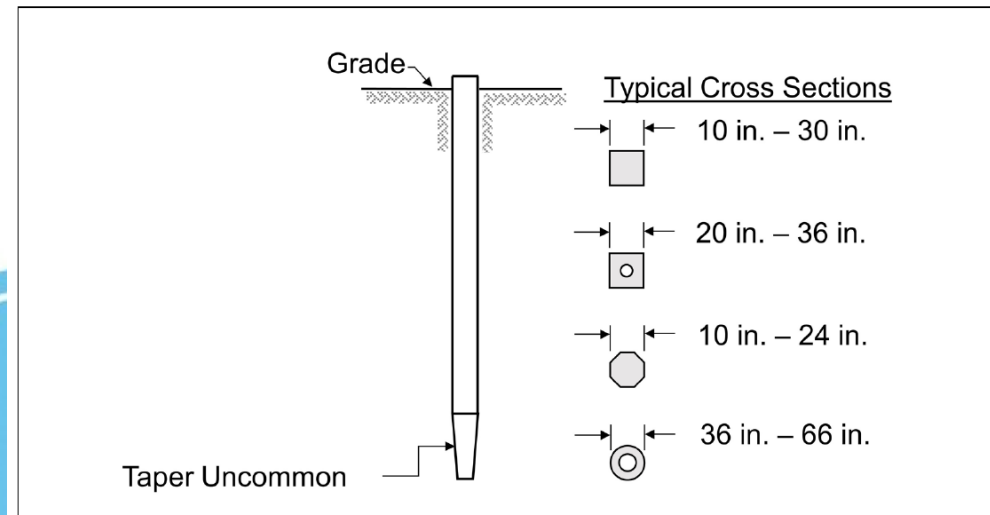


Figure 6-5 Precast, prestressed concrete typical illustration.

Square and Cylinder Pile Specifications

Table 2.4. Section properties and allowable service loads of prestressed concrete piles.

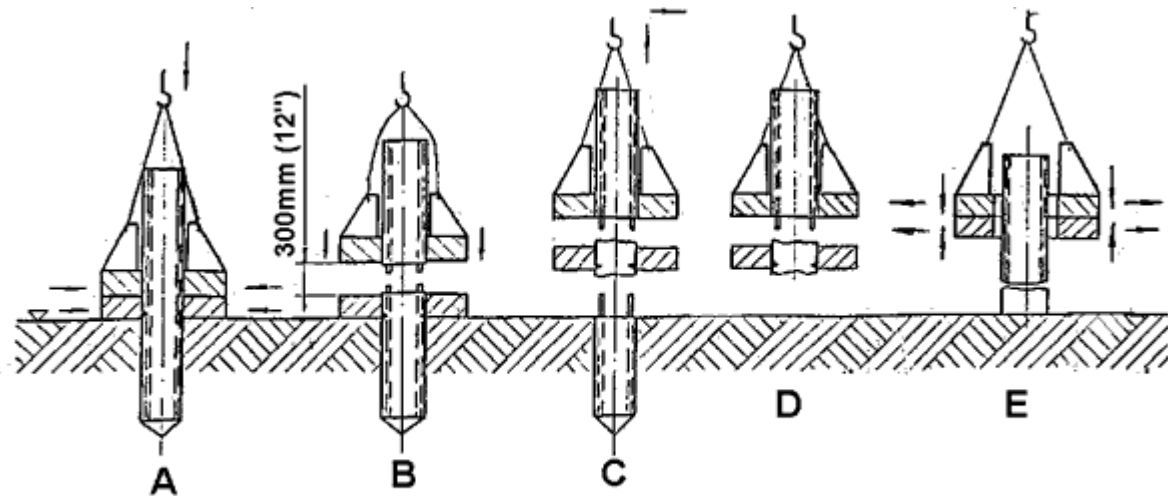
Size Continuous Tie Prestressing Strand* Square Solid Size Core Diameter Square Hollow Size Octagonal Solid or Hollow Size Round 5 turns @ 1" 16 turns @ 3" 6" pitch 16 turns @ 3" 5 turns @ 1" 1" </											
--	--	--	--	--	--	--	--	--	--	--	--

(1) Form dimensions may vary with producers, with corresponding variations in section properties.

(2) Allowable loads based on $N = A_c (0.33 f'_c + 0.27 f_{pc})$; $f_{pc} = 700$ psi. Check local producer for available concrete strengths.

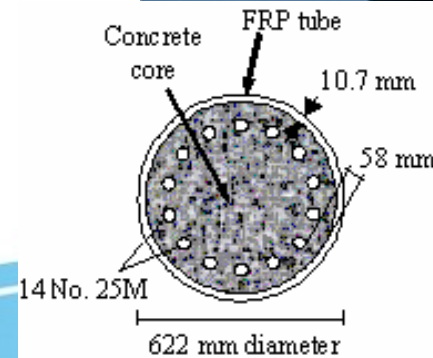
Cutting Concrete Piles

- Can be cut with concrete saws or special pile cutters
- Usually best to drive as friction piles where refusal is not as likely to take place



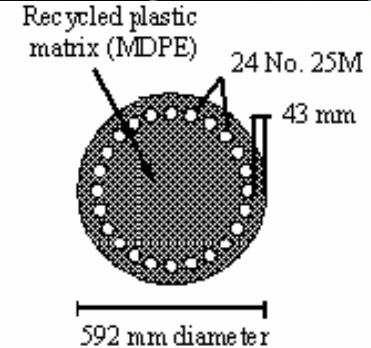
Composite Piles

- Two possible definitions
 - Pile made up of two other pile types, such as a concrete pile with an H-pile "stinger" on the end
 - Pile made up of two materials
- Plastic-steel composite
 - A useful substitute for wood piles in applications where wood is environmentally unacceptable



Long. rebar tied to a No. 9M external spiral with a 0.15 m pitch.

b) FRP pile



Long. rebar welded to a No. 9M internal spiral with a 0.23m pitch.

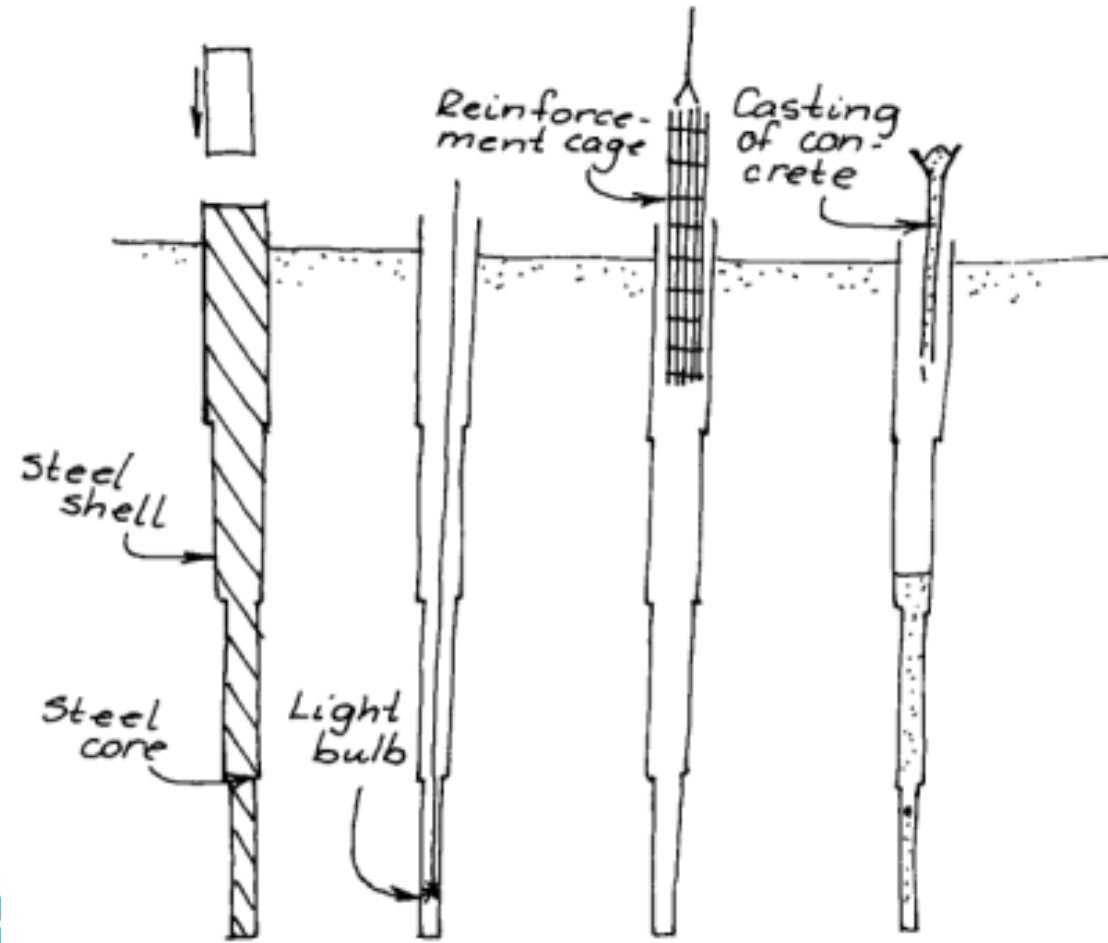
c) Plastic pile

1 mm = 0.039 inches

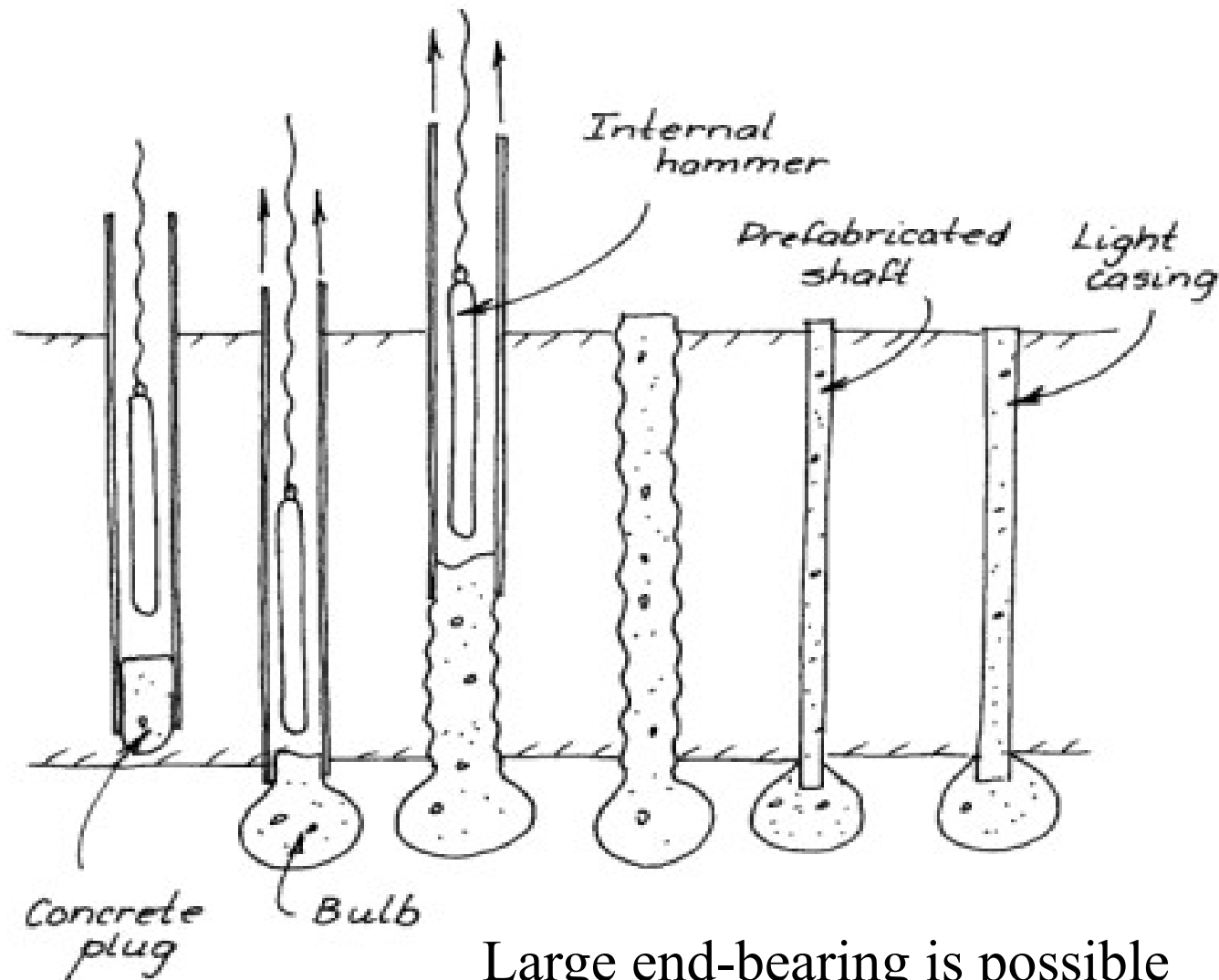
1 m = 3.28 ft

Mandrel-Driven Thin Shelled Piles

- Thin shelled steel piles which are driven with the assistance of a mandrel, as they would collapse if driven directly (as with pipe piles)
- Piles then filled with concrete and a reinforcing cage
- First widely popular mandrel driven thin shelled pile was the Raymond Step-Taper Pile, but there are many other kinds available today



Pressure-Injected Footings (Franki Piles)



Large end-bearing is possible

Typical Capacities and Lengths of Driven Pile Types

Table 9-2
Typical piles and their range of loads and lengths

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 (400-1,000 kN) for reinforced 90-1000 kips (400-4,500 kN) for prestressed	30-50 ft (10-15m) for reinforced 50-130 ft (15-40m) for prestressed
Steel H	130-560 kips (600-2,500 kN)	15-130 ft (5-40 m)
Steel Pipe (without concrete core)	180-560 kips (800-2,500 kN)	15-130 ft (5-40 m)
Steel Pipe (with concrete core)	560-3400 kips (2,500-15,000 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

Installation Equipment for Driven Piles

- Pile Driving Rigs
- Pile Hammers
- Hammer Accessories
 - Leaders
 - Cushion Material
- Predrilling, Jetting and Spudding

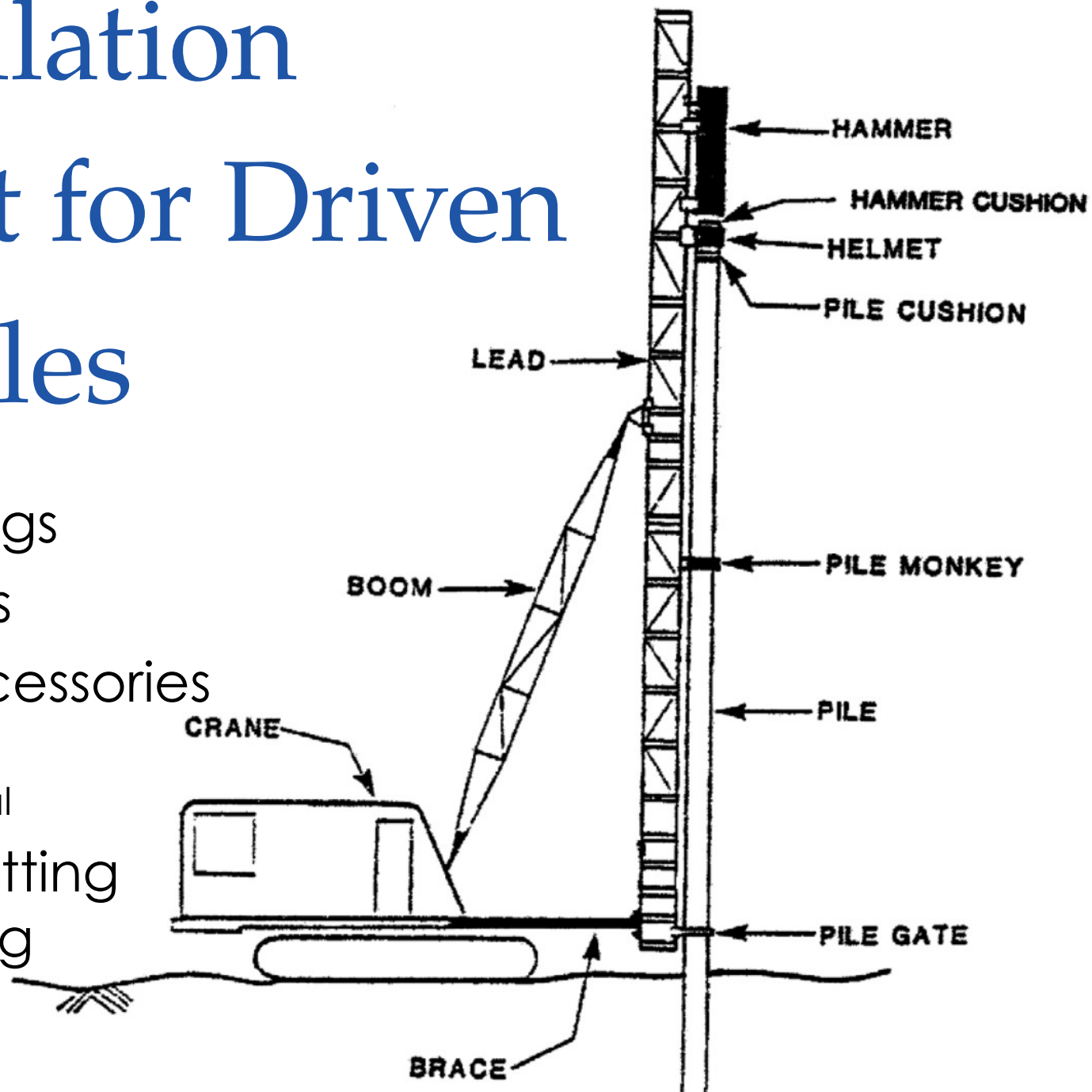


Figure 9-38. Typical components of a pile driving system.

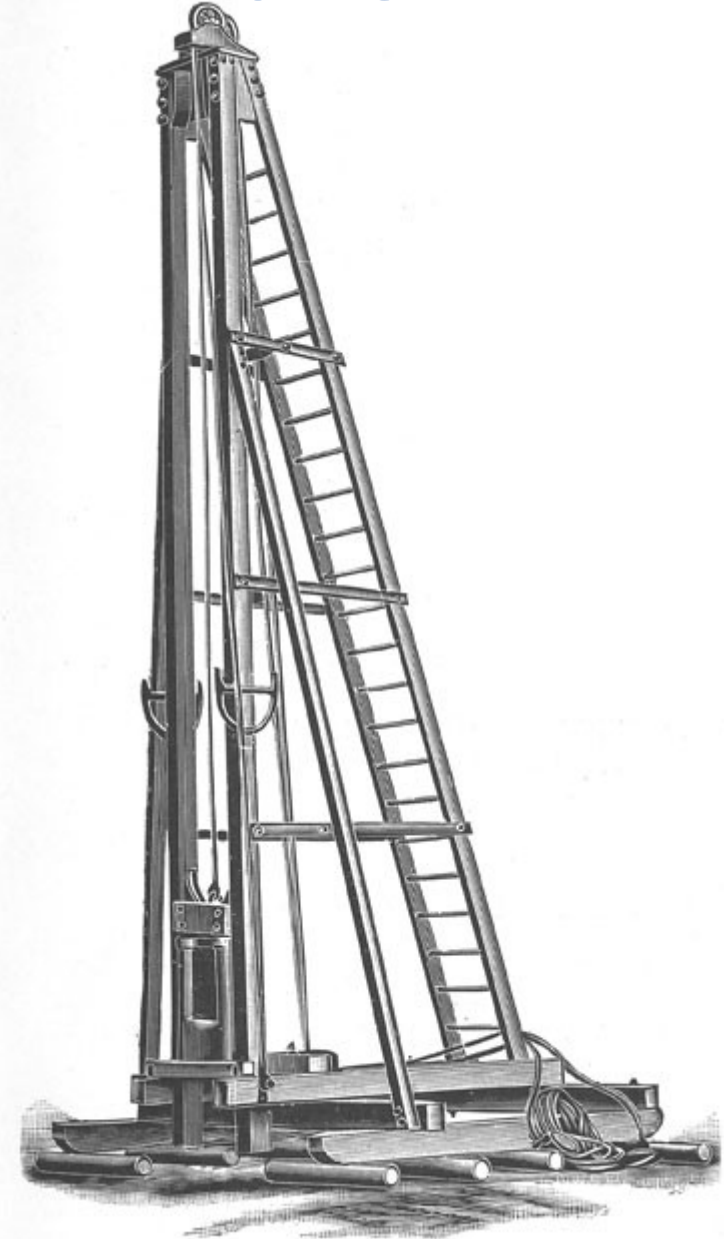
Pile Hammers

- Impact Hammers
 - Drop Hammers
 - Air/Steam Hammers
 - Diesel Hammers
 - Hydraulic Hammers
- Vibratory Hammers
- Pile Jacking Devices



Drop Hammers

- Oldest type of hammer in use
- Simply raised by the crane (or hoist) and released to impact the pile top
- A very simple hammer, yet slow and efficiency is inconsistent



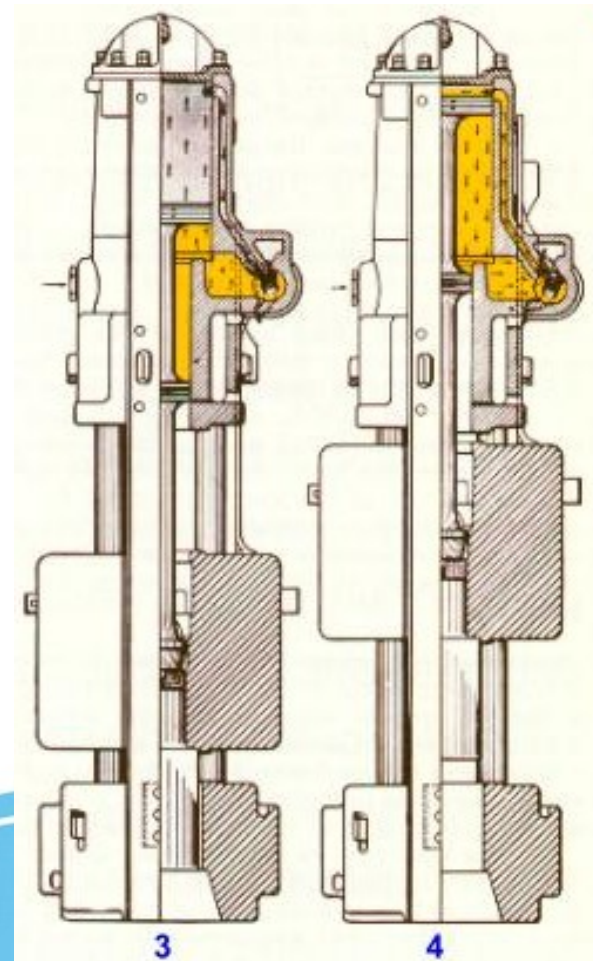
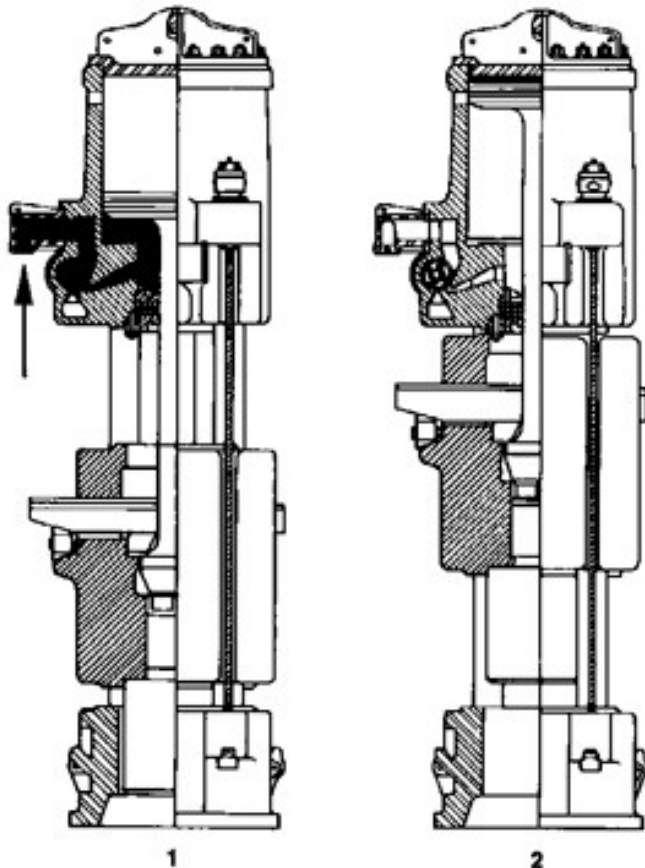
Air / Steam Hammers

- In use since the nineteenth century
- Hammers are simple; require little maintenance and are of long duration
- Efficiency also variable due to age of hammers and conditions of operation
- Hammers can be single, double or differential acting



Air / Steam Cycles

- Single-Acting (No Downward Assist)
- Differential or Double-Acting (Downward assist)



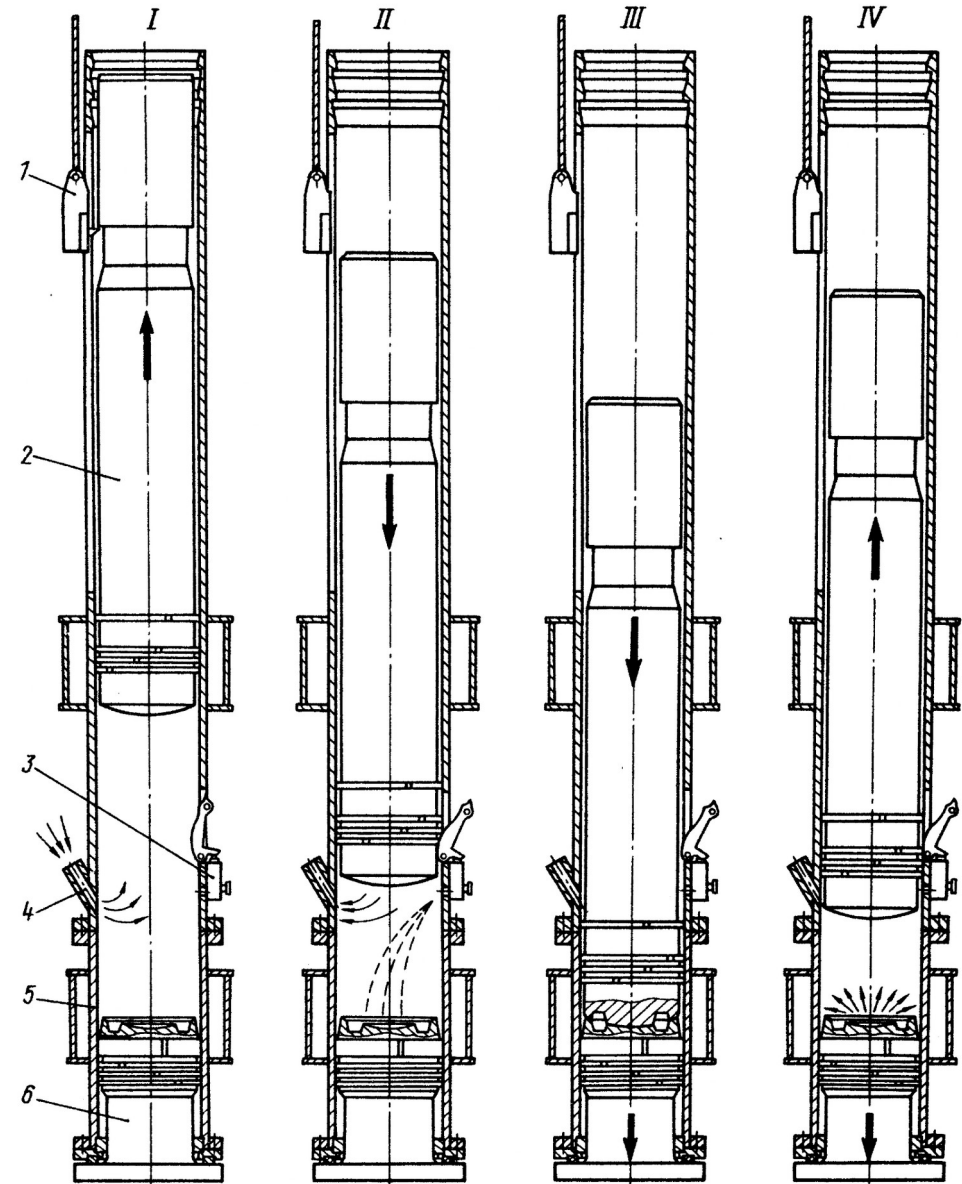
Diesel Hammers

- Developed in Germany between the two World Wars
- Does not require an external power source; usually light
- Can also be single-acting or double-acting



Operating Cycle of Diesel Hammers

- I. Upstroke or starting of the hammer with starting device (crab)
- II. Lowering of ram; injection of fuel
- III. Combustion at bottom of stroke
- IV. Fuel ignition and upward lifting of ram





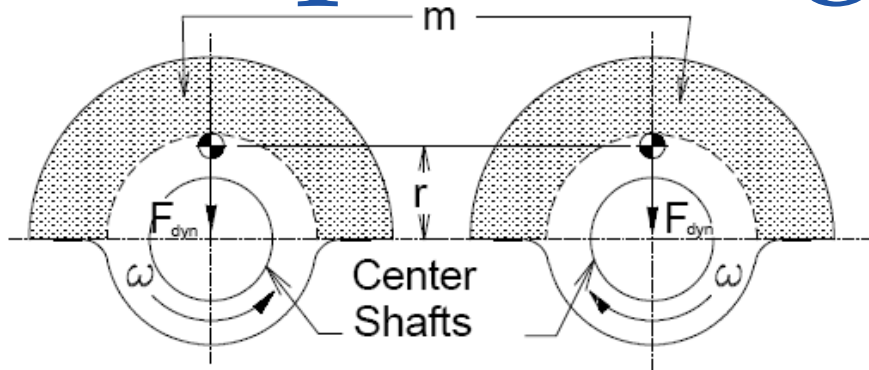
Hydraulic Impact Hammers



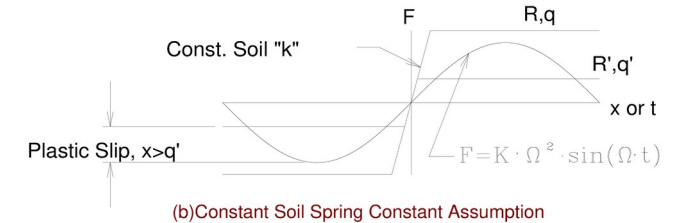
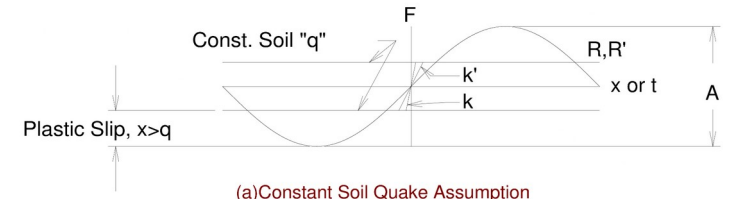
Vibratory Hammers



Operating Principle



- Vibratory hammers apply a rapidly alternating force to the pile by rotating eccentric weights about horizontal shafts
- Each eccentric produces centrifugal, dynamic force acting in a single plane and directed toward the centreline of the shaft
- The eccentrics are paired so the horizontal forces cancel each other, leaving only vertical force for the pile



Original Development and Soviet Equipment

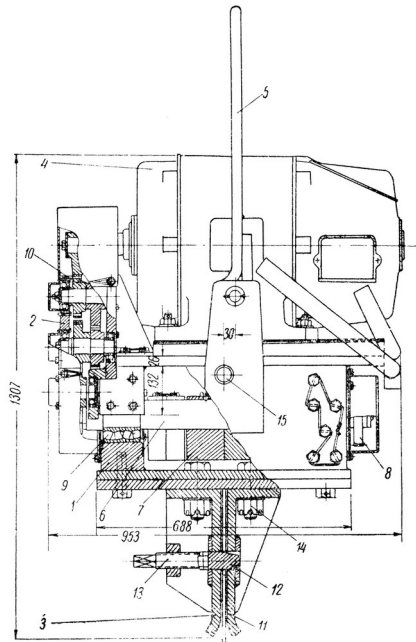
- First job in the USSR -- Gorki hydroelectric development, 1949

Model BT-5

Dynamic Force, 214 kN,
Eccentric Frequency, 41.67 Hz,
Power, 28 kW

Sheet Piles

3700 sheet piles, 9-12 m long, 2-3 minutes driving time



- Soviet B-402 pile driver
 - Dynamic force, 270 kN
 - Maximum eccentric moment, 12 kg-m
 - Rotation frequency, 23.8 Hz
- Driving sheet piling in Leningrad (St. Petersburg)

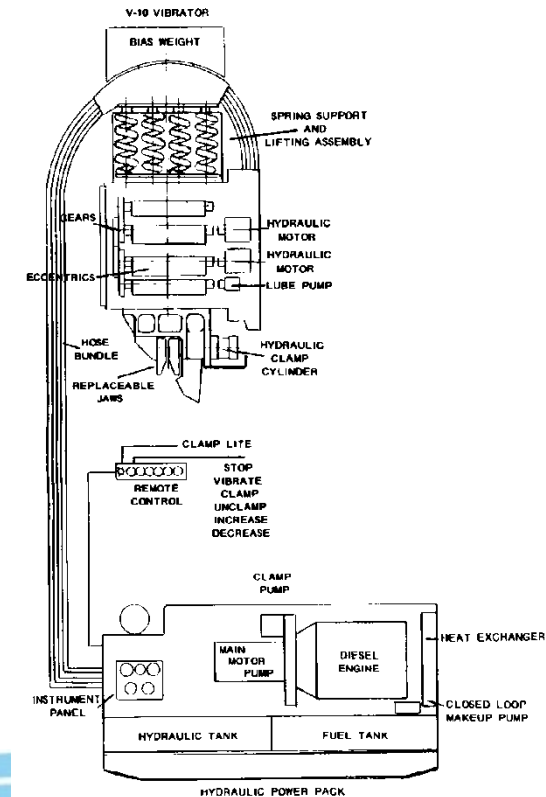


Other Vibratory Hammers

- Japan
 - Nippei
 - Uraga
 - Tomen
- France
 - PTC
- Germany
 - Müller
 - MGF

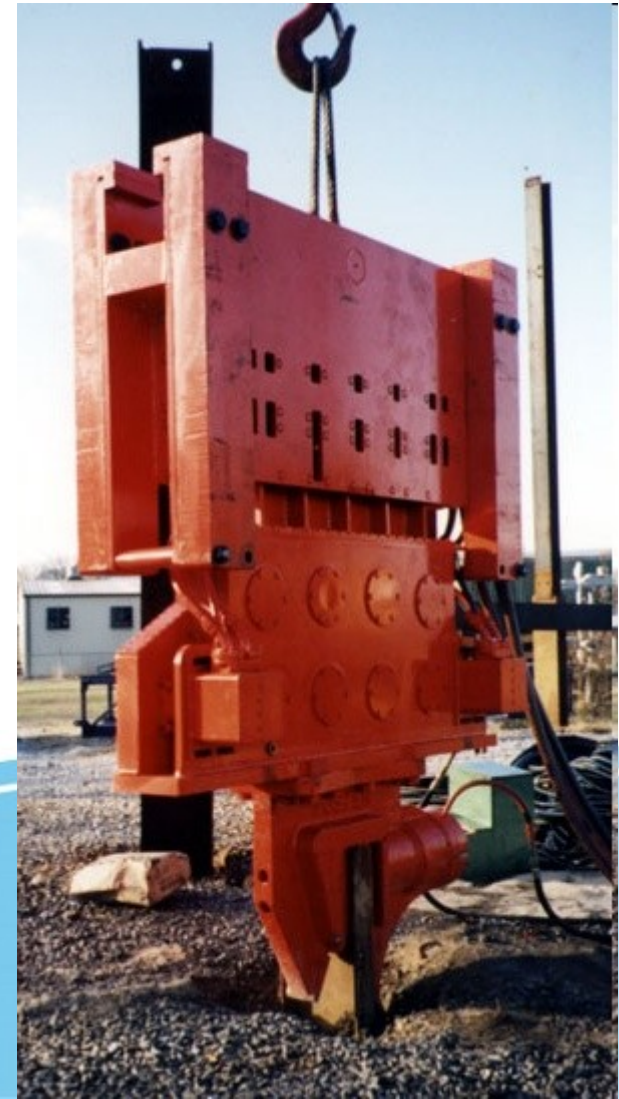


- U.S.
 - MKT – first U.S. Vibratory (V-10)
 - Foster (PTC, then Nippei derived)
 - ICE (US and Europe)
 - Vulcan
 - HPSI
 - APE



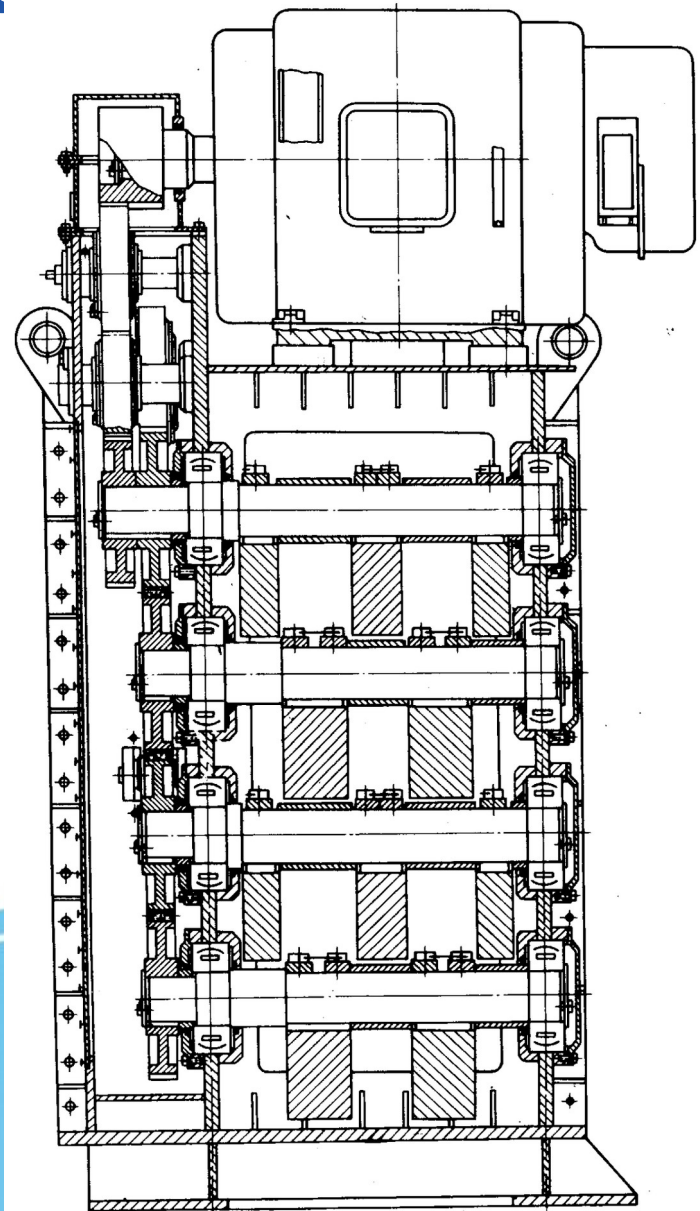
Basic Types of Vibratory Hammers

- Low frequency vibrators
- Medium frequency vibrators
- High frequency vibrators



Low Frequency Vibrators

- Characteristics
 - Vibration frequency of 5-10 Hz
 - Used with piles with high mass and toe resistance
 - Drive with high eccentric moments and amplitudes
- VPM-170
 - Dynamic force 1,700 kN
 - Frequency 9.17 hZ
 - Eccentric moment 510 kg-m



Medium Frequency Vibrators

- Characteristics
 - Frequency range – 10-30 Hz
 - Balance of frequency, eccentric moment and dynamic force needed to drive wide variety of piles
 - Most larger vibrators fall in this range



<http://www.youtube.com/watch?v=L5xFptZbfTw>

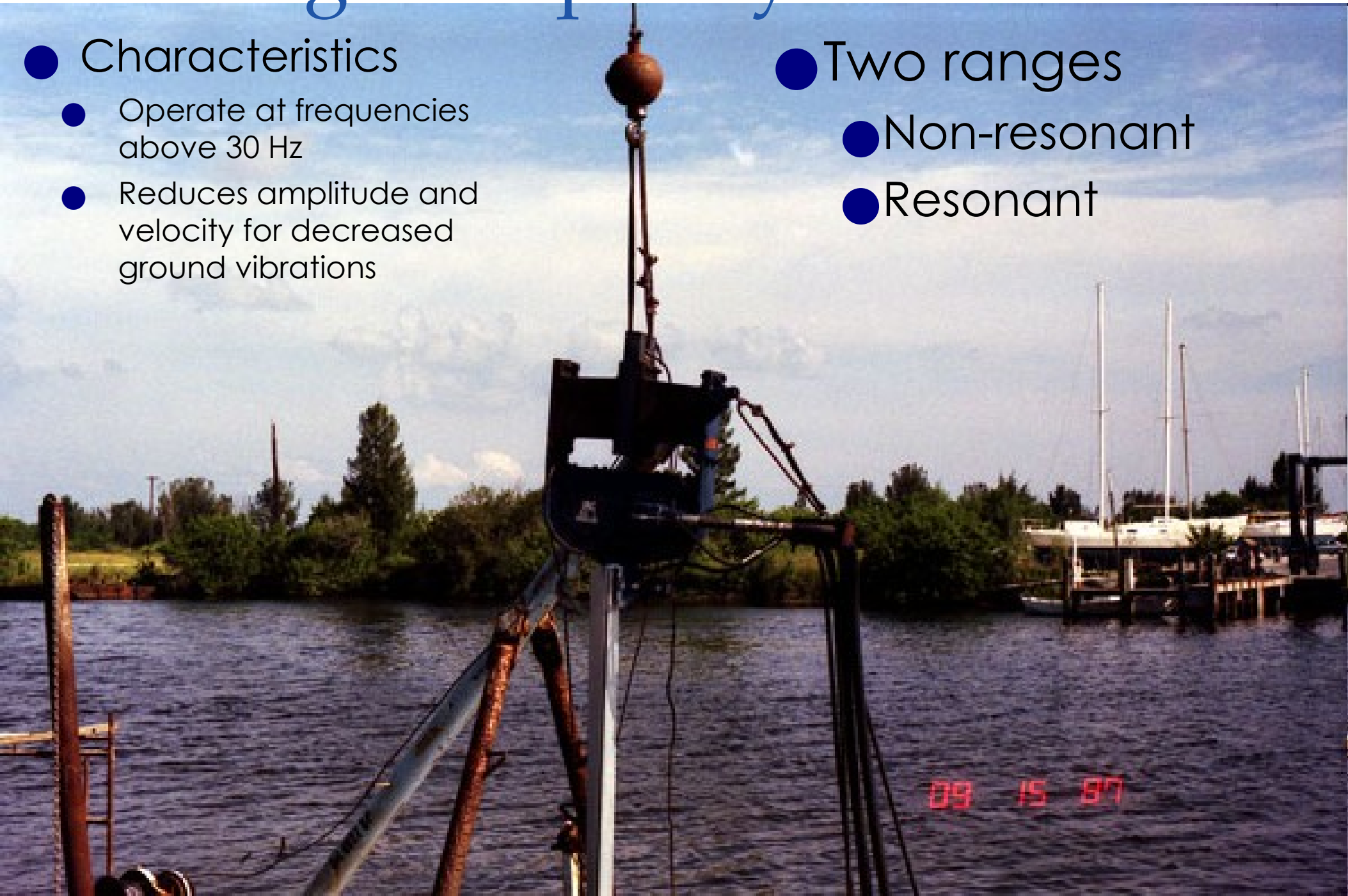
High Frequency Vibrators

- Characteristics

- Operate at frequencies above 30 Hz
- Reduces amplitude and velocity for decreased ground vibrations

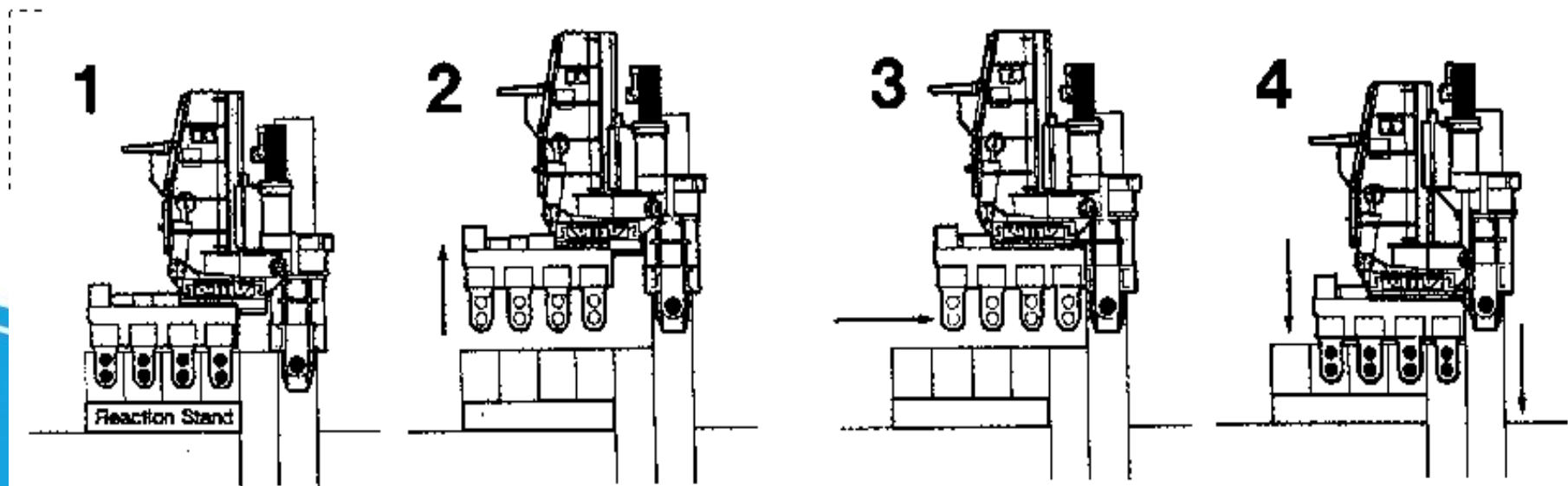
- Two ranges

- Non-resonant
- Resonant



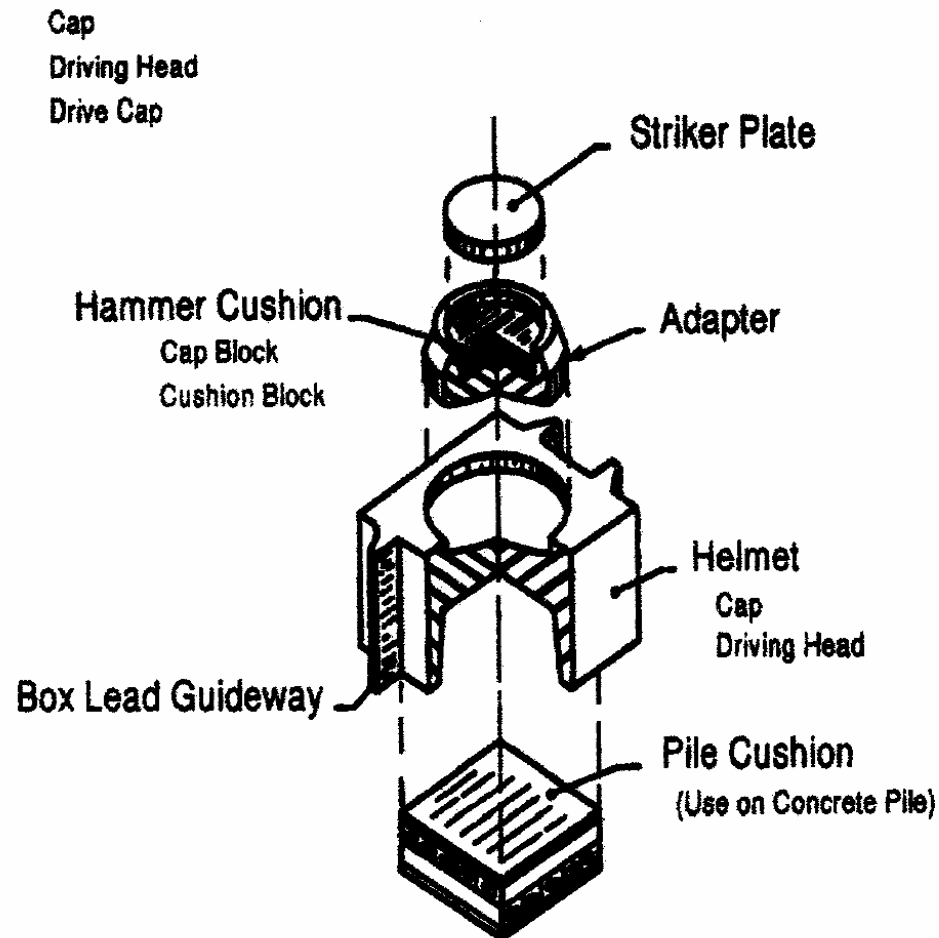
Pile Jacking Device

- Installs piling by pushing them into the soil, not impact
- Useful in situations where vibrations cannot be tolerated



Driving Accessories

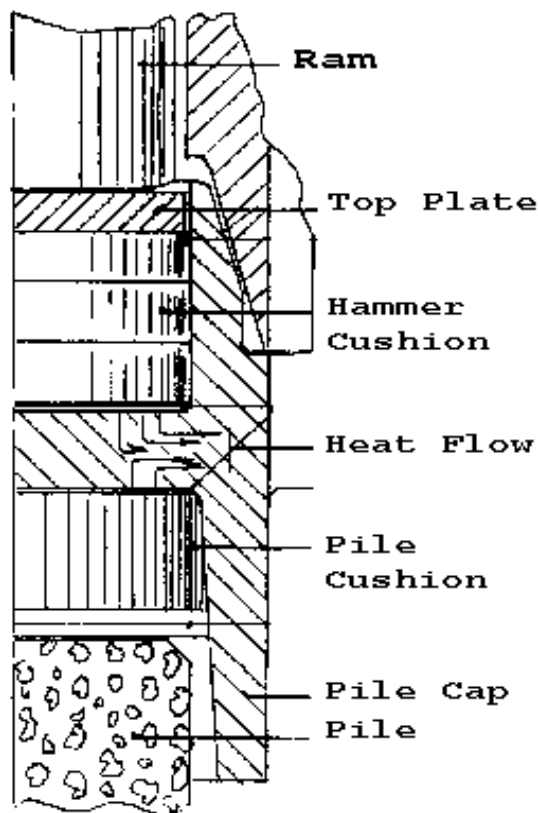
Helmet (Complete Unit)



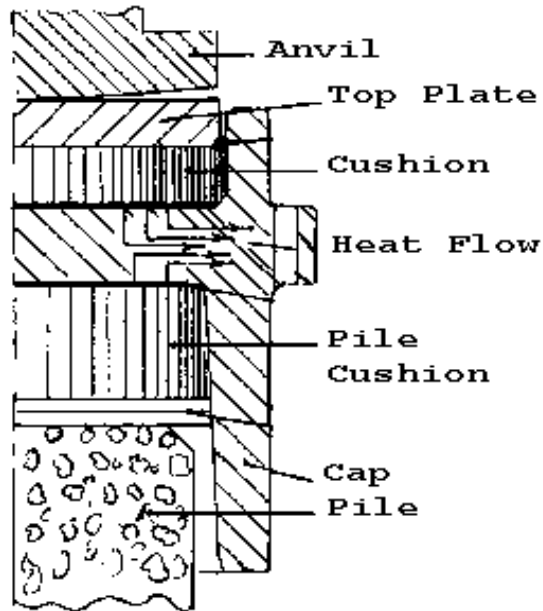
- Drive Caps
 - Mate the hammer to the pile
 - Hold the cushion material in place
- Weight of the cap influences the energy transmission from the hammer to the pile

Note: The helmet shown is for nomenclature only. Various sizes and types are available to drive H, pipe, concrete (shown) and timber piles. A system of inserts or adapters is utilized up inside of the helmet to change from size to size and shape to shape.

Cushion Material



Air-Steam Hammers
Integral Ring



Diesel Hammers

- Hammer Cushion
 - Protects the hammer and modulates the blow
 - Usually struck via a top plate above
- Pile Cushion
 - Used only with concrete piles
 - Protects the pile from cracking and spalling

Leaders

Fixed
Leaders



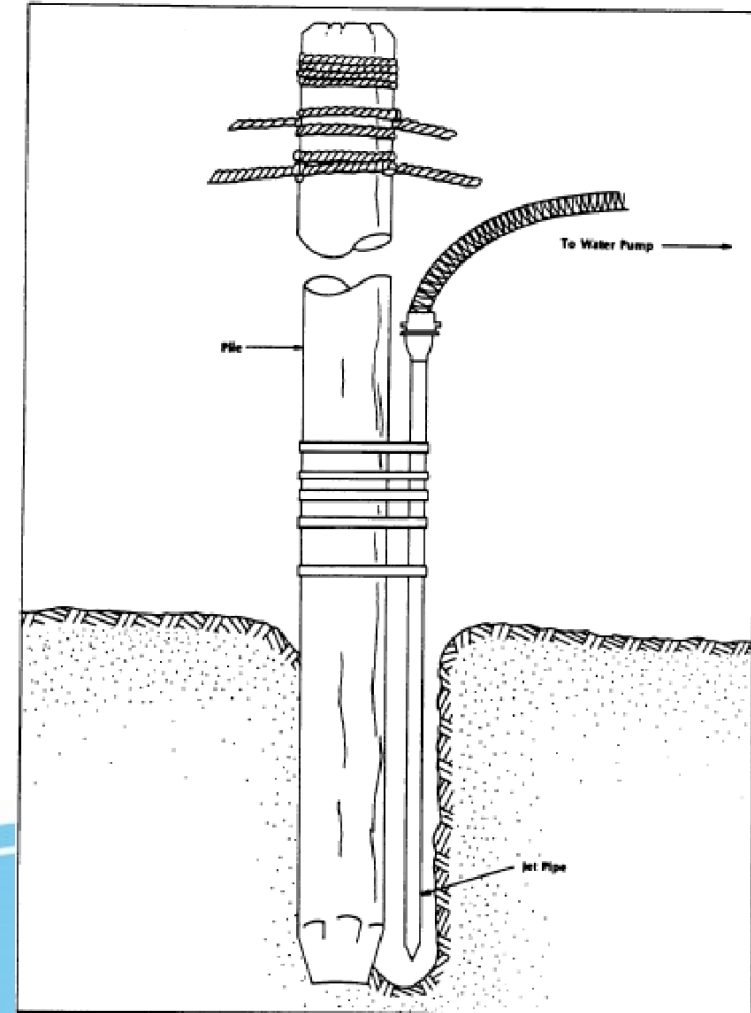
Swinging Leaders



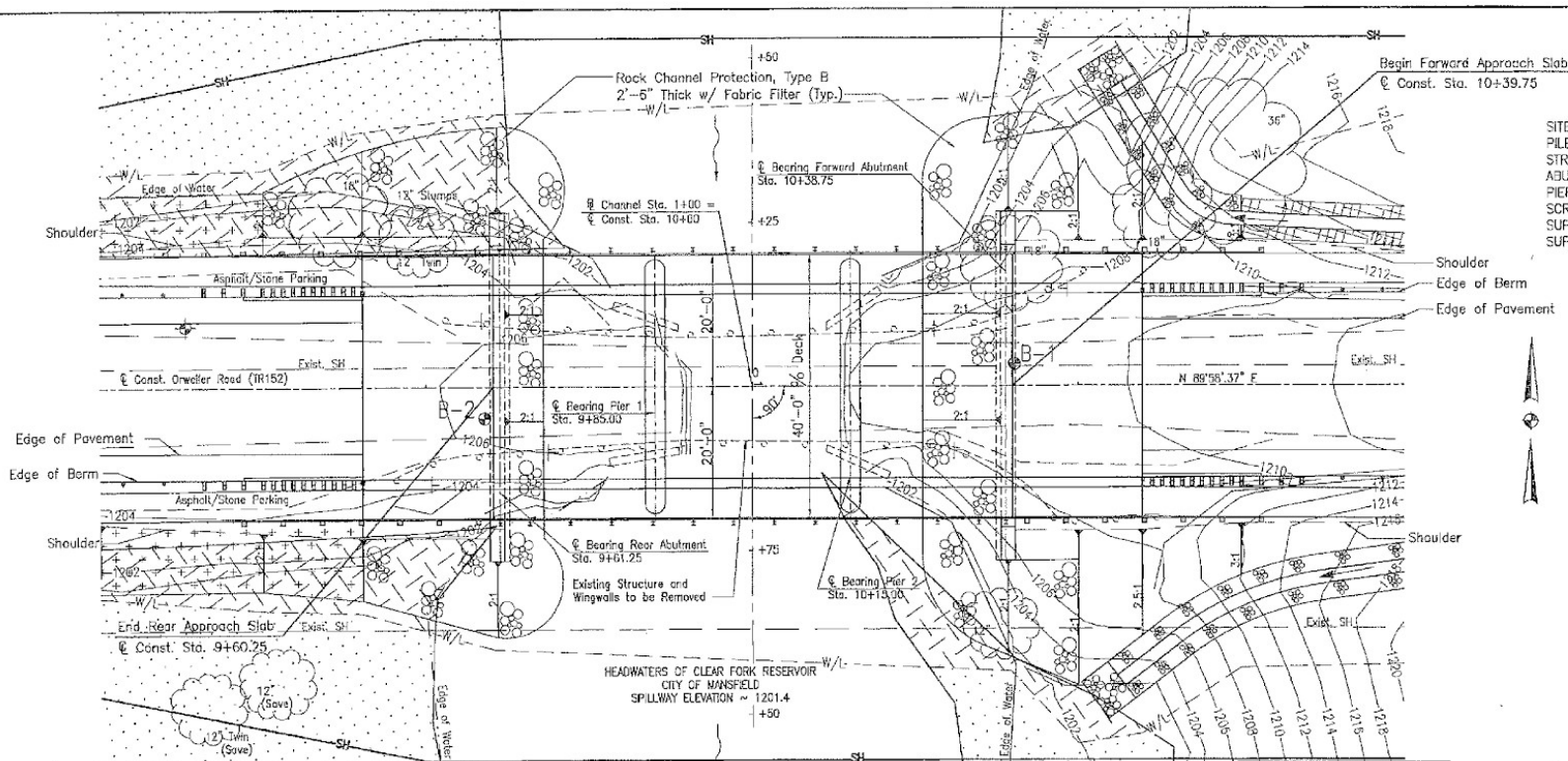
Predrilling and Jetting



- Methods of reducing the soil resistance to assist driving
 - Predrilling
 - Using an auger (usually continuous flight) to drill a hole into which the pile is driven
 - Does result in loss of shaft friction
 - Jetting



Axial Capacity of Driven Piles



INDEX OF SHEETS FOR STRUCTURE

SITE PLAN	1
PILE LAYOUT PLAN	2
STRUCTURE QUANTITIES & STRUCTURE NOTES	3
ABUTMENTS	4
PIERS	5
SCREED DIAGRAM & RAILING DETAILS	6
SUPERSTRUCTURE	7
SUPERSTRUCTURE REINFORCING STEEL & SECTIONS	8

Earthwork Limits shown are approximate. Actual slopes shall conform to plan cross sections.

LOCATION

USGS Blooming Grove Quadrangle, Richland County

N 40°43'32" (40.7255)
W 82°39'15" (-82.6542)

HYDRAULICS INFORMATION

DRAINAGE AREA = 11.9 Sq. Miles
Est. Q_{10} = 1460 cfs; V_{10} = 4.0 fps
Est. Q_{100} = 2530 cfs; V_{100} = 6.0 fps
Structure clears Est. 10-year H.W. by 6.5 ft. and Est. 100-year H.W. by 3.9 ft.
Roadway is not overlapped by the 100-year H.W.

FOUNDATION INVESTIGATION LEGEND

⊗ Indicates boring location.

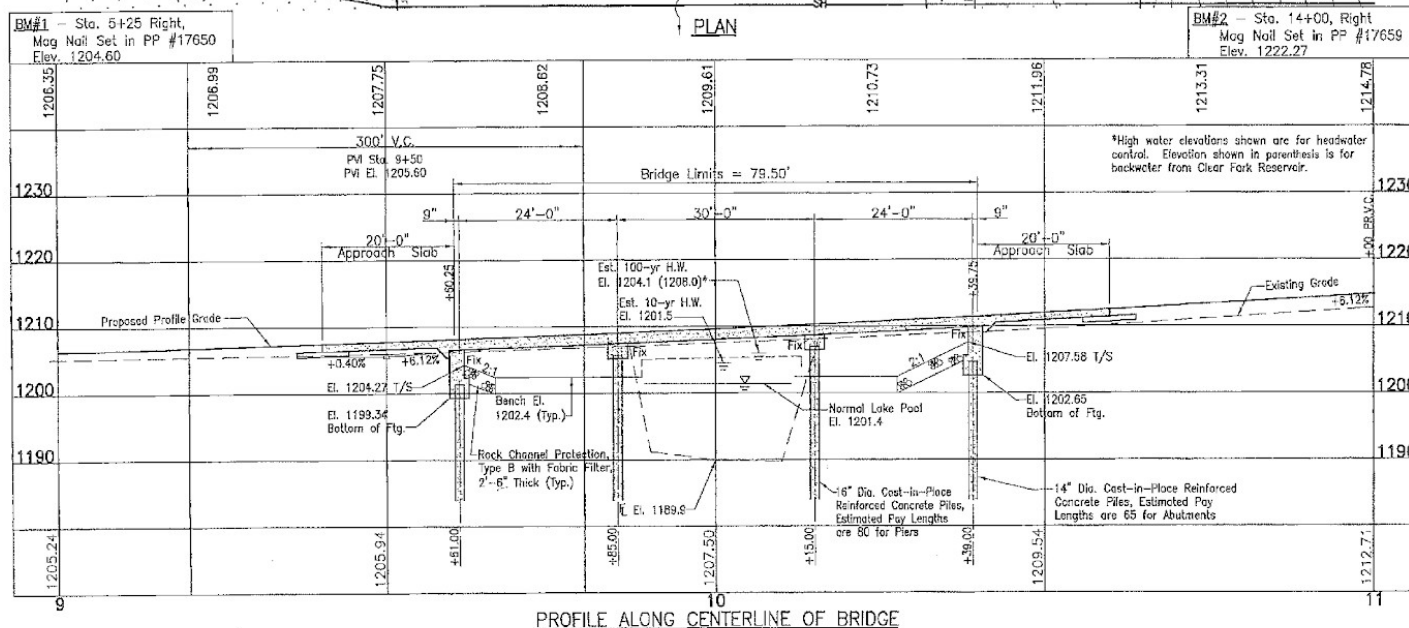
EXISTING STRUCTURE

TYPE A: Single-span steel beam with corrugated metal deck on concrete capped timber pile and stone abutments.

SPAN: 23'-0"
ROADWAY: 16'-0" % 15'-10" F_g Guardrail
LOADING: Unknown, Posted
SKEW: 0°
WEARING SURFACE: Asphalt Concrete
APPROACH SLABS: Possible Concrete
HEIGHT-GRADE TO STREAM BED: 18'+
ALIGNMENT: Tangent
STRUCTURE FILE NO.: 7032676
CONDITION: Poor (General Appraisal 4P)
DATE BUILT: 1950; Superstructure 1977
DISPOSITION: Remove

PROPOSED STRUCTURE

TYPE: Three-span Continuous Concrete Slab with Capped Pile Substructure.
SPAN: 24' ~ 30' ~ 24'
ROADWAY: 28'-0" Top, 1'-6" Deflector Parapets, 4'-6" Sidewalk, 40'-0" % Deck
LOADING: HL-93 + FWS of 60 lbs/ft²
SKEW: 0'-00"
WEARING SURFACE: Monolithic Concrete
APPROACH SLABS: AS-1-15 (20' Long)
ALIGNMENT: Tangent
NORMAL CROWN: 0.016
AVERAGE DAILY TRAFFIC: 250 (2039)
STRUCTURE FILE NO.: 7032677
HYDRAULIC DESIGN YEAR FREQUENCY: 10 year



Failure Methods of Deep Foundations

- Bearing Capacity
 - Catastrophic, limit-state failure of deep foundations (such as with slope stability and shallow foundations) is rare due to the difficulties of generating a “clean” failure surface
 - Nevertheless, we have an extensive collection of methods to estimate the “bearing capacity” of the pile
 - These can be used (with caution and generous safety/resistance factors) to estimate the point at which settlement is unacceptable

Settlement

Excessive settlement is the normal way we expect failure in deep foundations

Best way to anticipate excessive settlement is to model the load-deflection characteristics of the pile head

We can use information from “bearing capacity” methods to help this analysis

We can also model the soil mass around the pile as well

Must be verified in the field, either with static or dynamic load testing

Shaft and Toe Resistance

- The interaction between the pile and the soil allows the pile to resist the axial load
- This resistance is divided into two parts:
 - Shaft resistance, the shear resistance of the side surface of the pile to the downward movement of the pile during loading
 - Toe resistance, the compressive resistance of the end of the pile to downward movement
- The resistance of each is the product of the area of interaction with the soil and the ability of the soil to resist load
 - Shaft resistance: the “wetted” area, which is the perimeter of the pile times the length of the shaft
 - The shaft can be divided into layers, as soils vary with depth
 - Toe resistance: the cross-sectional area at the toe (sometimes different from the rest of the pile, sometimes the same) which faces the soil at the toe

● Static Method of Pile Design

The general equation for the ultimate axial capacity of driven piles is

$$Q_u = R_s + R_p$$

Where

- Q_u = ultimate axial capacity of the pile, kN
- R_s = ultimate shaft capacity of the pile, kN
- R_t = ultimate toe capacity of the pile, kN
- W_p = weight of the pile, kN

This is valid for piles loaded in compression; for tension piles, the toe capacity is not included and (in some cases) the weight of the pile is added.

The shaft capacity is in turn estimated by the equation

$$R_s = \int f(z)_z dA_z = \sum_{i=1}^n \overline{f_{s_n}} A_{s_n}$$

Where

- $f(z)_s$ = unit shaft friction along the pile shaft as a function of depth, kPa
- A_s = shaft area of the pile which interfaces with the soil, m²
- $\overline{f_{s_n}}$ = average shaft friction along a portion n of the pile, kPa
- A_{s_n} = shaft area of portion n of the pile, m²

This equation is only solved in the integral form in theoretical considerations. For practical considerations, it is solved in the summation form. Piles are customarily divided up into regions with a reasonably uniform soil type and unit shaft resistance.

The capacity of the pile toe is computed by the equation

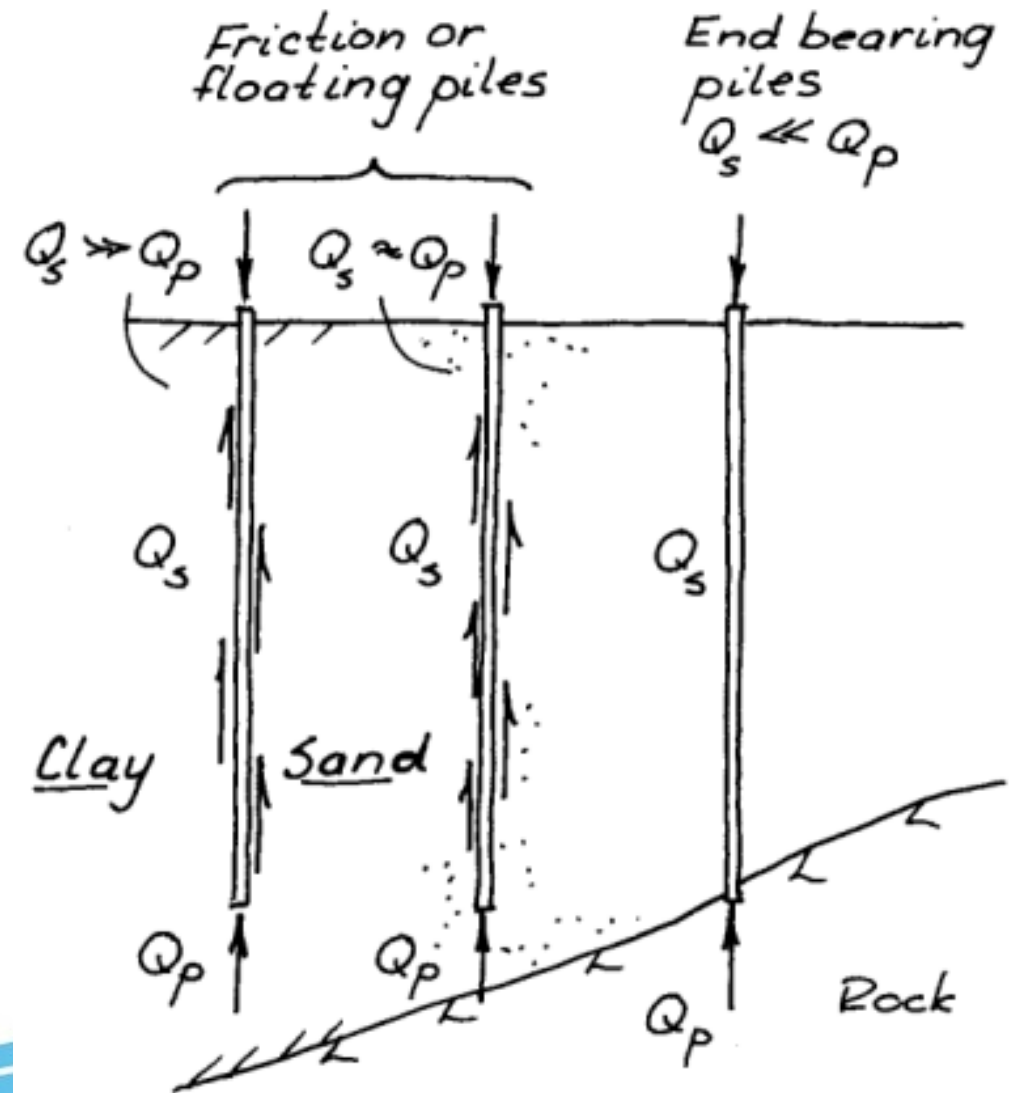
$$R_p = q_t A_t$$

Where

- q_t = unit pile toe capacity, kPa
- A_p = area of pile toe, m²

Load Transfer Characteristics of Piles

- Unlike shallow foundations, deep foundations transfer the load “progressively” to the soil during loading
- This is due to:
 - Flexibility of the pile
 - Variations in soil resistance along the pile and at the toe
- Diagram at the right shows these variations due to soil:
 - a) “Toe-bearing” pile, no shaft resistance
 - b) Typical cohesive soil
 - c) Typical cohesionless soil



Driven Pile Factors of Safety (ASD)

9.4.1 Factors of Safety

The results of static analyses yield a **geotechnical ultimate pile capacity**, Q_u . The **allowable geotechnical soil resistance (geotechnical pile design load)**, Q_a , is selected by dividing the **geotechnical ultimate pile capacity**, Q_u , by a **factor of safety** as follows.

$$Q_a = \frac{Q_u}{\text{Factor of Safety}} \quad 9-3$$

The range of the factor of safety, FS, has depended primarily upon the reliability of the particular method of static analysis with consideration of the following items:

1. The level of confidence in the input parameters. The level of confidence is a function of the type and extent of the subsurface exploration and laboratory testing of soil and rock materials.
2. Variability of the soil and rock.
3. Method of static analysis.
4. Effects of and consistency of the proposed pile installation method.
5. Level of construction control (static load test, dynamic analysis, wave equation analysis, Gates dynamic formula).

Table 9-5. Recommended factor of safety based on construction control method

Construction Control Method	Factor of Safety
Static load test with wave equation analysis	2.00
Dynamic testing with wave equation analysis	2.25
Indicator piles with wave equation analysis	2.50
Wave equation analysis	2.75
Gates dynamic formula	3.50

Driven Pile Capacity using Static Method

Consider a pile to be driven through the soil profile described in Figure 9-5. The proposed pile type penetrates through a sand layer subject to scour in the 100-year flood into an underlying very soft clay layer unsuitable for long term support and then into competent support materials. The soil resistances from the scour-susceptible sand layer and soft clay layer do not contribute to long term load support and should not be included in the soil resistance for support of the design load. In this example, static load testing with wave equation analysis will be used for construction control. Therefore, a factor of safety of 2.0 should be applied to the ultimate soil resistance calculated in suitable support layers in the static analysis. It should be noted that this approach is for scour conditions under the 100-year or overtopping flood events and that a different approach would apply for the superflood or 500-year event. For a superflood, a minimum factor of safety of 1.0 is used. This minimum factor of safety is determined by dividing the maximum pile load by the sum of the shaft and toe resistances available below scour depth.

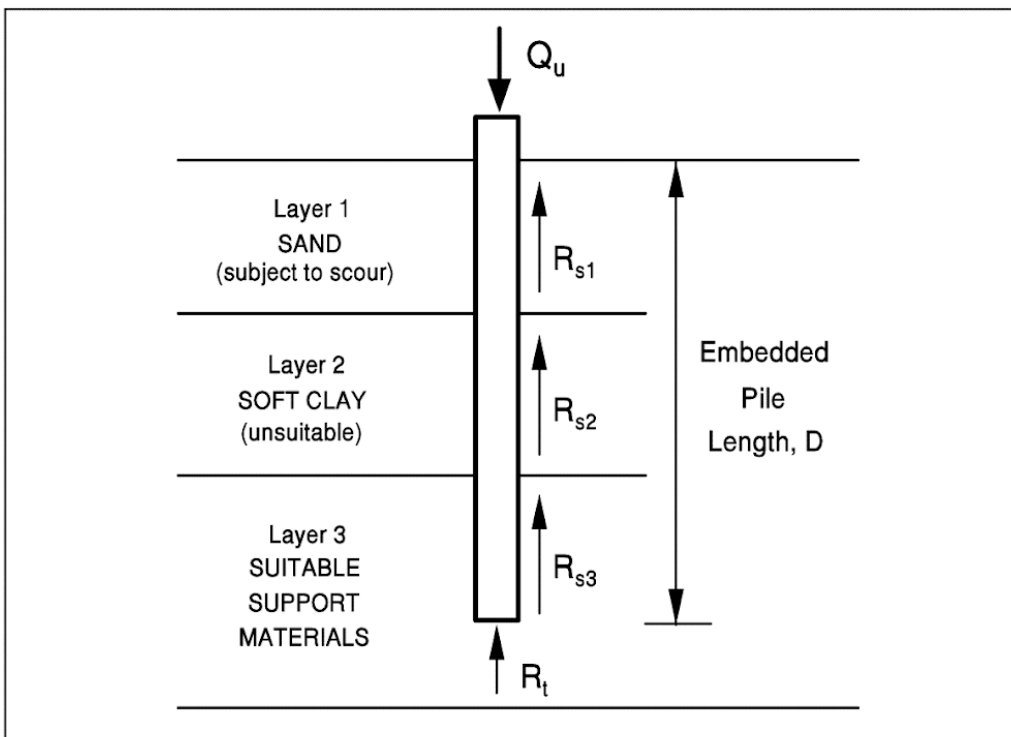


Figure 9-5. Soil profile for factor of safety discussion (FHWA, 2006a).

In the static analysis, a trial pile penetration depth is chosen and an ultimate pile capacity, Q_u , is calculated. This ultimate capacity includes the soil resistance calculated from all soil layers including the shaft resistance in the scour susceptible layer, R_{s1} , the shaft resistance in the unsuitable soft clay layer, R_{s2} as well as the resistance in suitable support materials along the pile shaft, R_{s3} , and at the pile toe resistance, R_t .

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

The design load, Q_a , is the sum of the soil resistances from the suitable support materials divided by a factor of safety, FS. As noted earlier, a factor of safety of 2.0 is used in the equation below because of the planned construction control with static load testing. Therefore,

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

The design load may also be expressed as the sum of the ultimate capacity minus the calculated soil resistances from the scour susceptible and unsuitable layers divided by the factor of safety. In this alternative approach, the design load is expressed as follows:

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

The result of the static analysis is then the estimated pile penetration depth, D , the design load for that penetration depth, Q_a , and the calculated ultimate capacity, Q_u .

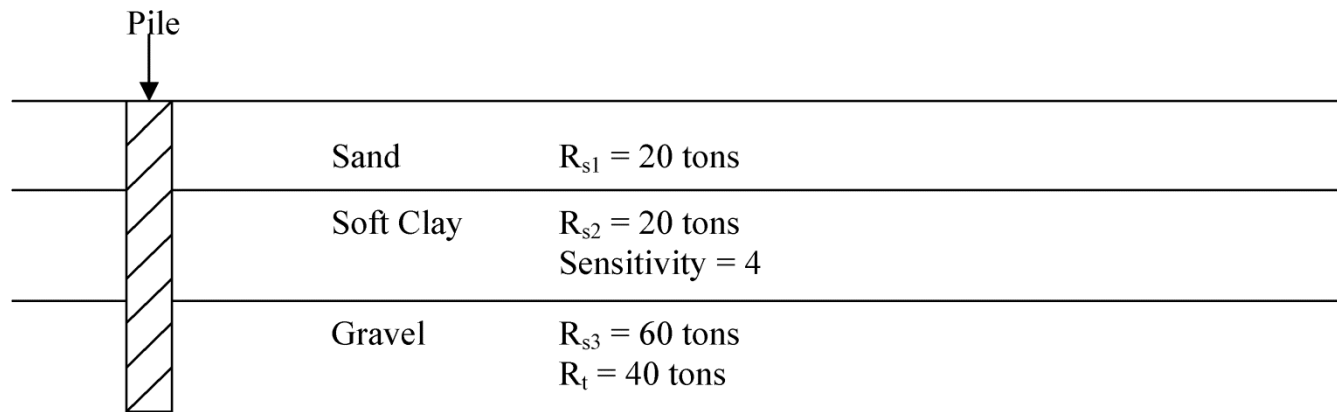
For preparation of construction plans and specifications, the **calculated geotechnical ultimate capacity**, Q_u , is specified. Note that if the construction control method changes after the design stage, the required ultimate capacity and the required pile penetration depth for the ultimate capacity will also change. This is apparent when the previous equation for the design load is expressed in terms of the ultimate capacity as follows:

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

Example of Pile Capacity

Using Static Method

Example 9-1: Find the ultimate capacity and driving capacity for the pile from the data listed in the profile. The hydraulic specialist determined that the sand layer is susceptible to scour. The geotechnical specialist determined that the soft clay layer is unsuitable for providing resistance.



Solution:

$$\begin{aligned}\text{Ultimate capacity} &= R_{s3} + R_t \\ &= 60 \text{ tons} + 40 \text{ tons} = 100 \text{ tons}\end{aligned}$$

$$\begin{aligned}\text{Driving capacity} &= R_{s1} + (R_{s2}/\text{Sensitivity}) + R_{s3} + R_t \\ &= 20 \text{ tons} + \frac{20 \text{ tons}}{4} + 60 \text{ tons} + 40 \text{ tons} = 125 \text{ tons}\end{aligned}$$

Methods for Static Analysis (from FHWA Driven Pile Manual)

Table 7-4 Methods of Static Analysis for Piles in Cohesionless Soils

Method	Approach	Method of Obtaining Design Parameters	Advantages	Disadvantages	Remarks
Nordlund Method.	Semi-Empirical.	Charts provided by Nordlund. Estimate of soil friction angle is needed.	Allows for increased shaft resistance of tapered piles and includes effects of pile-soil friction coefficient for different pile materials.	No limiting value on unit shaft resistance is recommended. Soil friction angle often estimated from SPT data. Limit on pile sizes.	Good approach to design that is widely used. Method is based on field observations. Details provided in Section 7.2.1.3.1.
API RP2A.	Empirical, effective stress analysis.	N_0 selected from Table 7-8 based on soil type.	Developed specifically for large diameter open end pipe.	Application to non-LDOEPs is limited.	Used almost exclusively for offshore pile design.
Effective Stress Method.	Semi-empirical.	β and N_t selected based on soil classification and estimated friction angle.	β value considers pile-soil friction coefficient for different pile materials. Soil resistance related to effective vertical stress.	Results affected by range in β values and in particular by range in N_t chosen.	Good approach for design. Details provided in Section 7.2.1.3.3.
Brown Method.	Empirical.	Results of SPT tests based on N_{60} values.	Widespread use of SPT test and input data availability. Simple method to use.	Relies solely on N_{60} values, which may not always be available.	Simple method based on correlations with 71 static load test results. Details provided in Section 7.2.1.3.5.
Methods based on Cone Penetration Test (CPT) data.	Empirical.	Results of CPT tests.	Testing analogy between CPT and pile. Reliable correlations and reproducible test data.	Limitations on pushing cone into dense strata.	Good approach for design. Details provided in Sections 7.2.1.3.6 and 7.2.1.3.7.

Table 7-5 Methods of Static Analysis for Piles in Cohesive Soils

Method	Approach	Method of Obtaining Design Parameters	Advantages	Disadvantages	Remarks
α -Method (Tomlinson Method).	Empirical, total stress analysis.	Undrained shear strength estimate of soil is needed. Adhesion calculated from Figures 7-17 and 7-18.	Simple calculation from laboratory undrained shear strength values to adhesion.	Wide scatter in adhesion versus undrained shear strengths in literature. Limits on s_u strengths in soft and medium cohesive soils.	Widely used method described in Section 7.2.1.3.2.
API RP2A.	Empirical, effective stress analysis.	Undrained shear strength estimate of soil is needed.	Developed specifically for large diameter open end pipe piles.	Application to non-LDOEPs is limited.	Used almost exclusively for offshore pile design.
Effective Stress Method.	Semi-Empirical, based on effective stress at failure.	β and N_t values are selected from Table 7-9 based on drained soil strength estimates.	Ranges in β and N_t values for most cohesive soils are relatively small.	Range in N_t values for hard cohesive soils such as glacial tills can be large.	Good design approach theoretically better than undrained analysis. Details in Section 7.2.1.3.3.
Methods based on Cone Penetration Test data.	Empirical.	Results of CPT tests.	Testing analogy between CPT and pile. Reproducible test data.	Cone can be difficult to advance in very hard cohesive soils such as glacial tills.	Good approach for design. Details in Section 7.2.1.3.6 and Section 7.2.1.3.7.

Fellenius Method

- Based on a concept originally set forth by John Burland in the early 1970's
 - Cohesive soils, especially when disturbed, act like cohesionless ones in that the resistance they offer to load is dependent upon effective stress

9.5.2.2 Effective Stress – β -method

Static capacity calculations in cohesionless, cohesive, and layered soils can also be performed by using an effective stress based method. Effective stress based methods were developed to model the long term drained shear strength conditions. Therefore, the effective soil friction angle, ϕ' , should be used in parameter selection.

In an effective stress analysis, the unit shaft resistance is calculated from the following expression:

$$f_s = \beta p_o \quad 9-12$$

where:

- β = Bjerrum-Burland beta coefficient = $K_s \tan \delta$.
- p_o = average effective overburden pressure along the pile shaft, in ksf (kPa).
- K_s = earth pressure coefficient.
- δ = interface friction angle between pile and soil.

The unit toe resistance is calculated from:

$$q_t = N_t p_t \quad 9-13$$

where:

- N_t = toe bearing capacity coefficient.
- p_t = effective overburden pressure at the pile toe in ksf (kPa).

Recommended ranges of β and N_t coefficients as a function of soil type and ϕ' angle from Fellenius (1991) are presented in Table 9-7. Fellenius (1991) notes that factors affecting the β and N_t coefficients consist of the soil composition including the grain size distribution, angularity and mineralogical origin of the soil grains, the original soil density and density due to the pile installation technique, the soil strength, as well as other factors. Even so, β coefficients are generally within the ranges provided and seldom exceed 1.0.

Fellenius Method

Table 9-7
Approximate range of β and N_t coefficients (Fellenius, 1991)

Soil Type	ϕ'	β	N_t
Clay	25 – 30	0.23 - 0.40	3 - 30
Silt	28 – 34	0.27 - 0.50	20 - 40
Sand	32 – 40	0.30 - 0.60	30 - 150
Gravel	35 – 45	0.35 - 0.80	60 - 300

For sedimentary cohesionless deposits, Fellenius (1991) states that N_t ranges from about 30 to a high of 120. In very dense non-sedimentary deposits such as glacial tills, N_t can be much higher, but it can also approach the lower bound value of 30. In clays, Fellenius (1991) notes that the toe resistance calculated by using an N_t of 3 is similar to the toe resistance calculated from an analysis where undrained shear strength is used. Therefore, the use of a relatively low value of the N_t coefficient in clays is recommended unless local correlations suggest higher values are appropriate.

Graphs of the ranges in β and N_t coefficients versus the range in ϕ' angle as suggested by Fellenius are presented in Figure 9-17 and 9-18, respectively. These graphs may be helpful in selection of β or N_t . The inexperienced user should select conservative β and N_t coefficients. As with any design method, the user should also confirm the appropriateness of a selected β or N_t coefficient in a given soil condition with local correlations between static capacity calculations and static load tests results.

It should be noted that the effective stress method places no limiting values on either the shaft or toe resistance.

STEP BY STEP PROCEDURE FOR THE EFFECTIVE STRESS METHOD

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

- Construct p_0 diagram by using previously described procedures in Chapter 2.
- Divide soil profile throughout the pile penetration depth into layers and determine the effective overburden pressure, p_0 , in ksf (kPa) at the midpoint of each layer.
- Determine the ϕ' angle for each soil layer from laboratory or in-situ test data.

- In the absence of laboratory or in-situ test data for cohesionless layers, determine the average corrected SPT N_1 value for each layer and estimate ϕ' angle from Table 8-1 in Chapter 8.

STEP 2 Select the β coefficient for each soil layer.

- Use local experience to select β coefficient for each layer.
- In the absence of local experience, use Table 9-7 or Figure 9-17 to estimate the β coefficient from the ϕ' angle for each layer.

STEP 3 For each soil layer compute the unit shaft resistance, f_s in ksf (kPa).

$$f_s = \beta p_0$$

STEP 4 Compute the shaft resistance in each soil layer and the ultimate shaft resistance, R_s in kips (kN) from the sum of the shaft resistance from each soil layer.

$$R_s = \sum f_s A_s$$

where: A_s = pile-soil surface area in ft^2 (m^2) = (pile perimeter) $\times$ (length).

STEP 5 Compute the unit toe resistance, q_t in ksf (kPa).

$$q_t = N_t p_t$$

- Use local experience to select N_t coefficient.
- In the absence of local experience, estimate N_t from Table 9-7 or Figure 9-18 based on ϕ' angle.
- Calculate the effective overburden pressure at the pile toe, p_t in ksf (kPa).

STEP 6 Compute the ultimate toe resistance, R_t in kips (kN).

$$R_t = q_t A_t$$

where: A_t = area of the pile toe in m^2 (ft^2).

Fellenius Method

STEP 7 Compute the ultimate geotechnical pile capacity, Q_u in kips (kN).

$$Q_u = R_s + R_t$$

STEP 8 Compute the allowable geotechnical soil resistance, Q_a in kips (kN).

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

The factor of safety in this static calculation should be based on the specified construction control method as described in Section 9.4 of this chapter. Recommended factors of safety based on construction control methods are listed in Table 9-5

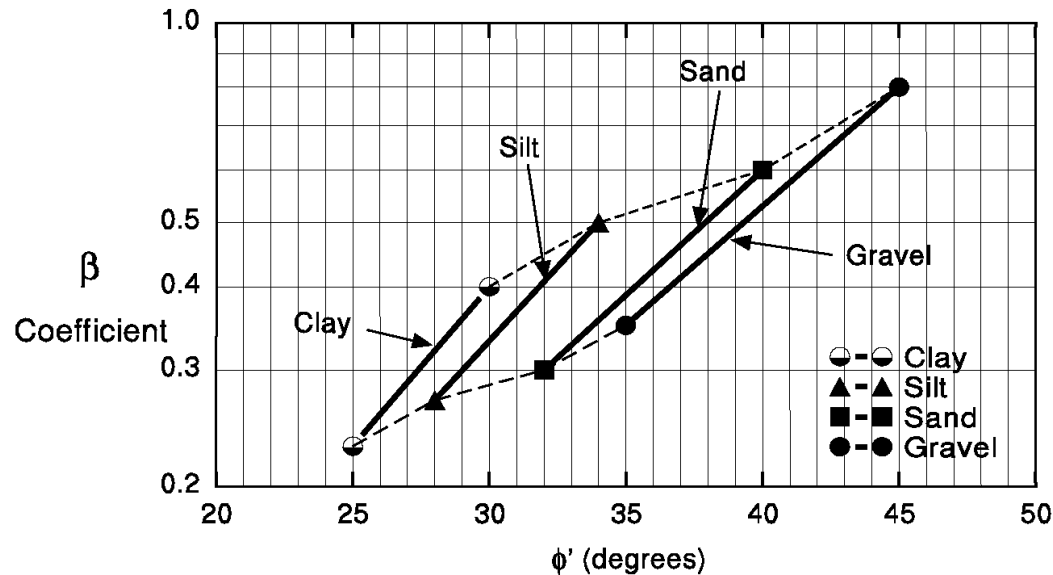


Figure 9-17. Chart for estimating β coefficient as a function of soil type ϕ' (after Fellenius, 1991).

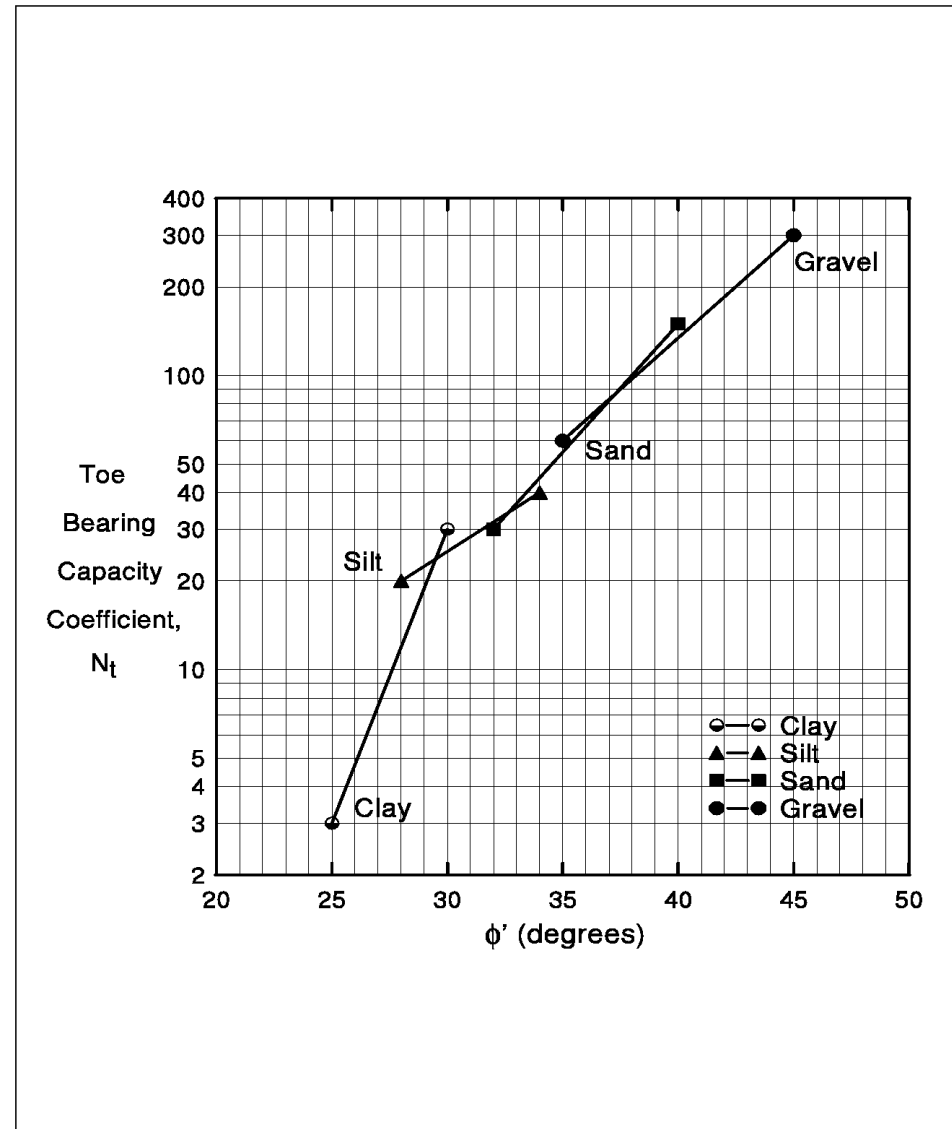
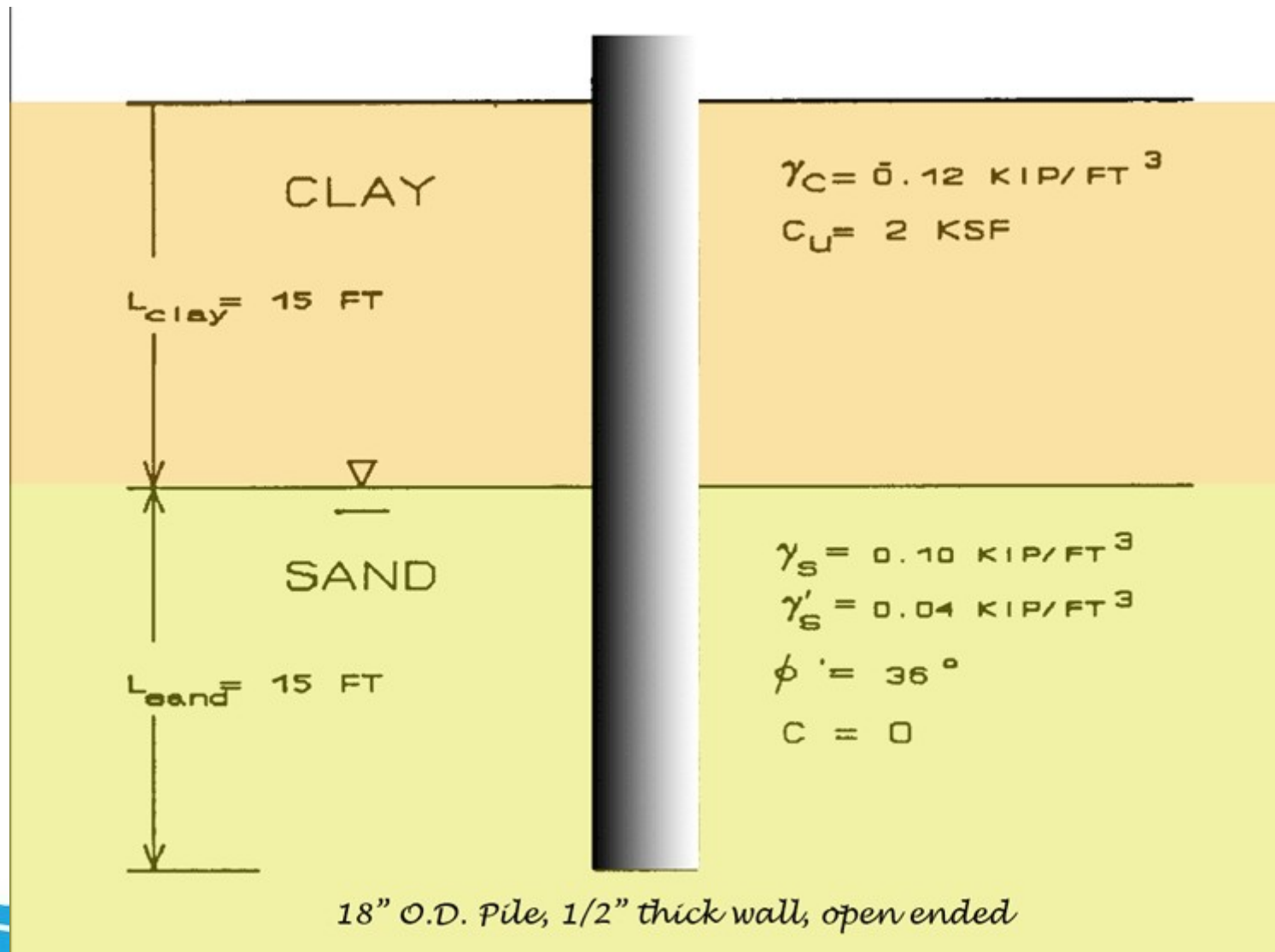


Figure 9-18. Chart for estimating N_t coefficients as a function of soil type ϕ' angle (after Fellenius, 1991).

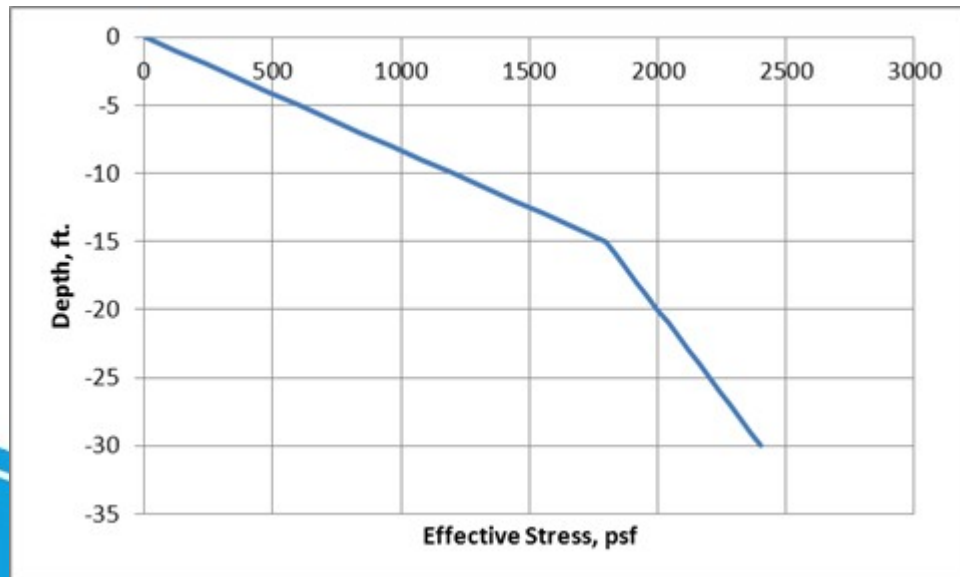
Example of Fellenius Method



Find: ultimate static capacity of pile using Fellenius Method. Assume no plugging in pile toe and static load testing for verification.

Example of Fellenius Method

- Solution
 - There are two layers, one in clay and one in sand
 - Since this is a β method, the first thing we do is to compute the effective stresses in all of the layers



- Effective Stresses
 - Effective Stress at Clay/Sand Layer Boundary = 1800 psf
 - Average Effective Stress in Clay Layer = 900 psf
 - Effective Stress at Pile Toe = 2400 psf
 - Average Effective Stress in Sand Layer = 2100 psf

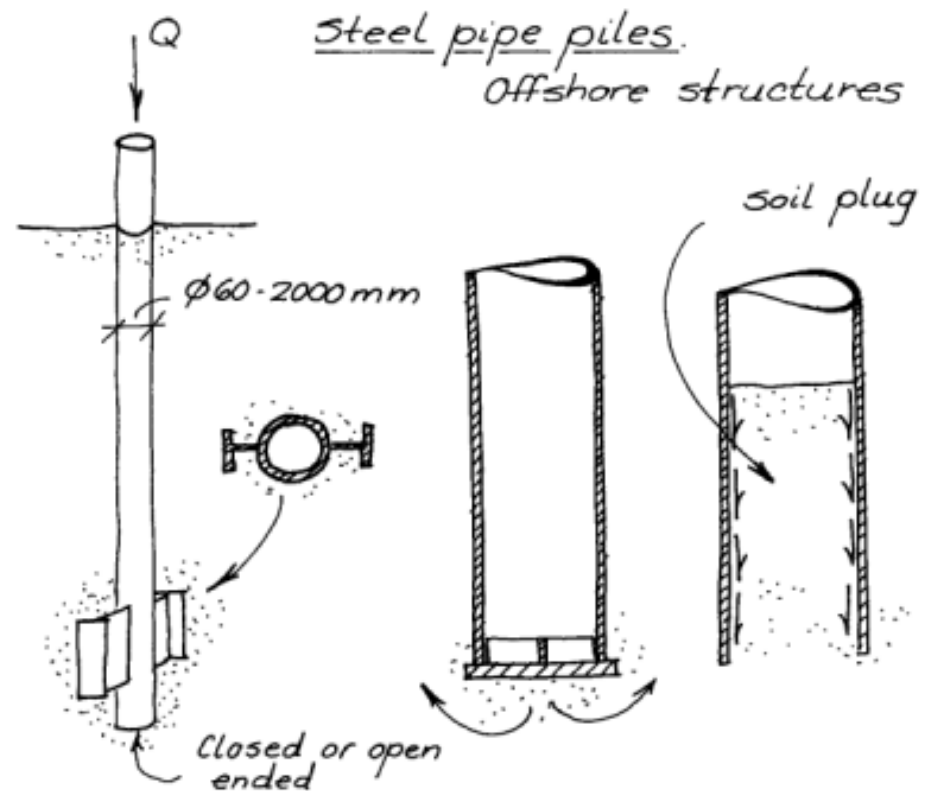
Example of Fellenius Method

- Shaft and Toe Areas
 - The soil has two layers (the water table is conveniently at the boundary of the two)
 - The pile diameter (18"), shape (circular,) and layer thickness are the same for both layers, thus the shaft area for each layer is the same, thus

$$A_s = \frac{\pi(18)(15)}{12} = 70.69 \text{ ft}^2$$

- Because we assume there is no soil plugging, the toe area is the same as the cross-sectional area of the pile, thus

$$A_{p-open} = \frac{(\pi)(18^2 - (18 - (2)(0.5))^2)}{(4)(144)} = 0.191 \text{ ft}^2$$



Example of Fellenius Method

● Solution

– We then need to compute Fellenius' β factors for the shaft

- For the top layer, assume $\beta = 0.23$ (Table 9-7)
- For the bottom layer, assume $\beta = 0.46$ (Table 9-7)

● Shaft Layer Unit Resistances

- Layer 1: $f_1 = (0.23)(900) = 207 \text{ psf} = 0.207 \text{ ksf}$
- Layer 2: $f_2 = (0.46)(2100) = 966 \text{ psf} = 0.966 \text{ ksf}$

Note: values of β can vary with different presentations of the method; the author has revised these over the years. These are based on the values in the SFH.

Example of Fellenius Method

- Solution

- Now we need to compute the shaft resistance for each layer and sum them up for the total shaft resistance

- Layer 1: $R_1 = f_1 A_1 = (0.207 \text{ ksf})(70.69 \text{ ft}^2) = 14.6 \text{ kips}$
- Layer 2: $R_2 = f_2 A_2 = (0.966 \text{ ksf})(70.69 \text{ ft}^2) = 68.3 \text{ kips}$
- Total shaft Resistance $R_t = 14.6 + 68.3 = 82.9 \text{ kips}$

- Toe Resistance

- There is only one toe; we only need to compute its resistance for the one layer it is embedded in
- That layer is the sand layer and the effective stress at the toe is 2.4 ksf
- The value of N_t for the toe is ~90 (Table 9-7)
- The unit shaft resistance $q_t = (90)(2.4) = 216 \text{ ksf}$
- The toe resistance $Q_t = (216)(0.191) = 41.3 \text{ kips}$

- Total Ultimate Resistance $= R_t + Q_t = 82.9 + 41.3 = 124.2 \text{ kips}$

Example of Fellenius Method

- Total Ultimate Resistance $Q_u = R_t + Q_t = 82.9 + 41.3 = 124.2$ kips
- The factor of safety we use depends upon the verification method
- Since we called for static load testing, $FS = 2$ (see table below)
- Thus, $Q_{allow} = 124.2/2 = 62.1$ kips

Table 9-5. Recommended factor of safety based on construction control method

Construction Control Method	Factor of Safety
Static load test with wave equation analysis	2.00
Dynamic testing with wave equation analysis	2.25
Indicator piles with wave equation analysis	2.50
Wave equation analysis	2.75
Gates dynamic formula	3.50

Profiles and Contact Areas

● Contact Areas

- Closed-section foundations
 - where foundation-soil contact takes place along a well defined border between pile and soil
 - Most deep foundations come under this designation
- Open Section Foundations
 - Where foundation-soil contact is poorly defined and the soil will move as the pile penetrates
 - This includes open-ended pipe piles and (to some degree) H-beams
 - This phenomenon is referred to as plugging

● Displacement Characteristics

Table 9-3

Pile type selection pile shape effects

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

Plugging in Open-Section Foundations



The above studies suggest that plugging in any soil material is probable under static loading conditions once the penetration-to-pile-diameter ratio exceeds 20 in dense sands and clays, or 20 to 30 in medium sands. An illustration of the difference in the soil resistance mechanism that develops on a pipe pile with an open and plugged toe condition is presented in Figure 9-19. Paikowsky and Whitman (1990) recommend that the static capacity of an open end pipe pile be calculated from the lesser of the following equations:

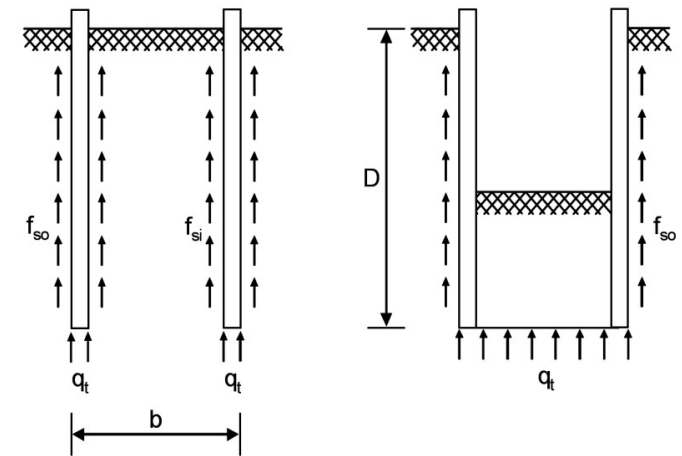
Plugged Condition:
$$Q_u = f_{so} A_s + q_t A_t \quad 9-15a$$

Unplugged Condition:
$$Q_u = f_{so} A_s + f_{si} A_{si} + q_t A_p - w_p \quad 9-15b$$

where: Q_u = ultimate pile capacity in kips (kN).
 f_{so} = exterior unit shaft resistance in ksf (kPa).

A_s = pile exterior surface area in ft^2 (m^2)
 f_{si} = interior unit shaft resistance in ksf (kPa)
 A_{si} = pile interior surface area in ft^2 (m^2)
 q_t = unit toe resistance in ksf (kPa)
 A_t = toe area of a plugged pile in ft^2 (m^2)
 A_p = cross sectional area of an unplugged pile in ft^2 (m^2)
 w_p = weight of the plug in kips (kN)

Static pile capacity calculations for open end pipe piles in cohesionless soils should be performed by using the Paikowsky and Whitman (1990) equations. Toe resistance should be calculated by using the Tomlinson limiting unit toe resistance of 105 ksf (5000 kPa), once Meyerhof's limiting unit toe resistance, determined from Figure 9-14, exceeds 105 ksf (5000 kPa). For open end pipe piles in predominantly cohesive soils, the Tomlinson equation should be used.



(a) Open Toe Condition

(b) Plugged Toe Condition

Figure 9-19. Plugging of open end pipe piles (after Paikowsky and Whitman, 1990).

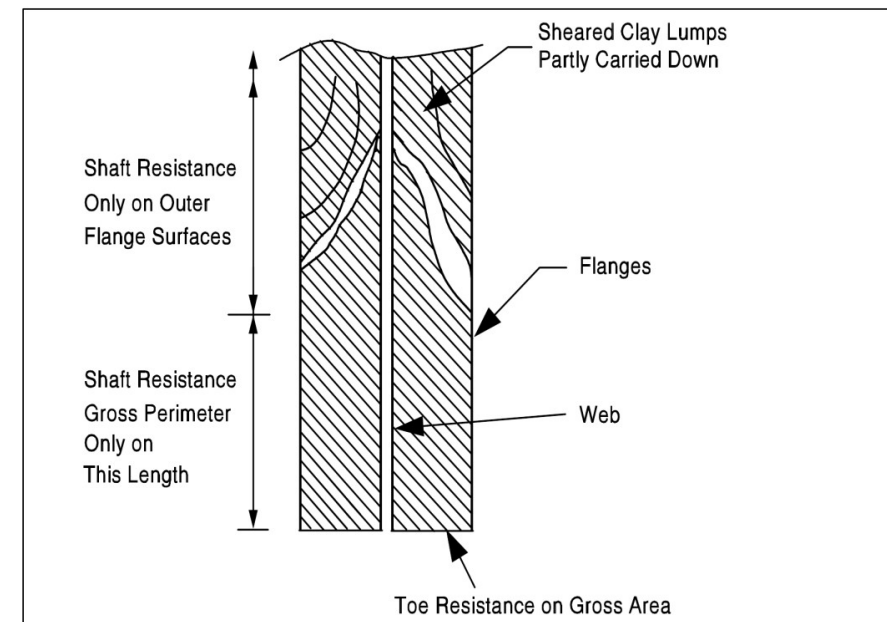


Figure 9-20. Plugging of H-piles (FHWA, 2006a).

Soil Set-Up

FHWA (1996) calculated general soil setup factors based on the predominant soil type along the pile shaft. The **soil setup factor** was defined as the failure load from a static load test divided by the end-of-drive wave equation capacity. These results are presented in Table 9-20. The data base for this study was comprised of 99 test piles from 46 sites. The number of sites and the percentage of the data base in a given soil condition is included in the table. While these soil set-up factors may be useful for preliminary estimates, soil setup is better estimated based on site-specific data gathered from pile restriking, dynamic measurements, static load testing, and local experience.

Komurka, *et al.*, (2003) summarized the current practice in estimating and measuring soil setup in a report to the Wisconsin Highway Research Program. This report summarizes the mechanisms associated with soil setup development and reviews several empirical relationships for estimating set-up.

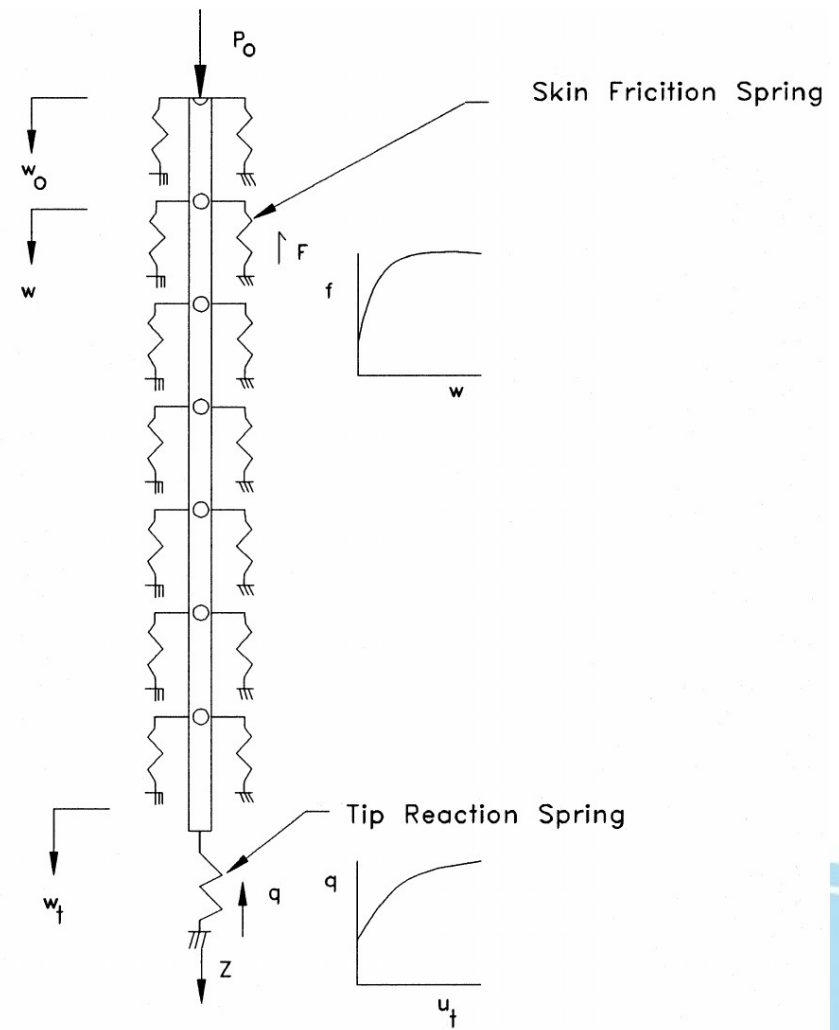
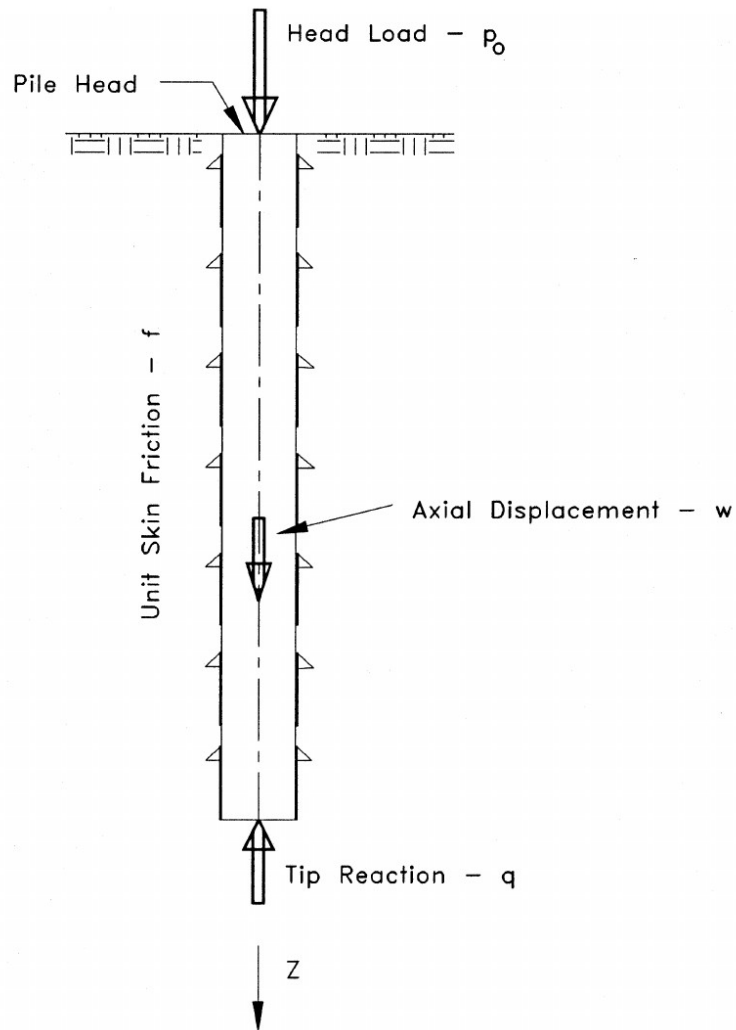
Table 9-8
Soil setup factors (after FHWA, 1996)

Predominant Soil Type Along Pile Shaft	Range in Soil Set-up Factor	Recommended Soil Set-up Factors*	Number of Sites and (Percentage of Data Base)
Clay	1.2 - 5.5	2.0	7 (15%)
Silt - Clay	1.0 - 2.0	1.0	10 (22%)
Silt	1.5 - 5.0	1.5	2 (4%)
Sand - Clay	1.0 - 6.0	1.5	13 (28%)
Sand - Silt	1.2 - 2.0	1.2	8 (18%)
Fine Sand	1.2 - 2.0	1.2	2 (4%)
Sand	0.8 - 2.0	1.0	3 (7%)
Sand - Gravel	1.2 - 2.0	1.0	1 (2%)
* Confirmation with local experience recommended			

Settlement

- Pile axial capacity is developed by “mobilizing” the shaft and toe resistance
 - Without pile deflection, there is no axial resistance to loading
 - Pile capacity thus cannot be considered without considering pile settlement
 - As with shallow foundations, settlement must ultimately be the main analysis
- In the case of “critical” structures, settlement analysis will be performed using a “t-z” method computer program
 - Example of one is in the TAMWAVE program
 - Another is the ALP program, which is available

t-z Method of Axial Load and Settlement Analysis



● Settlement Example

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

● Settlement Example

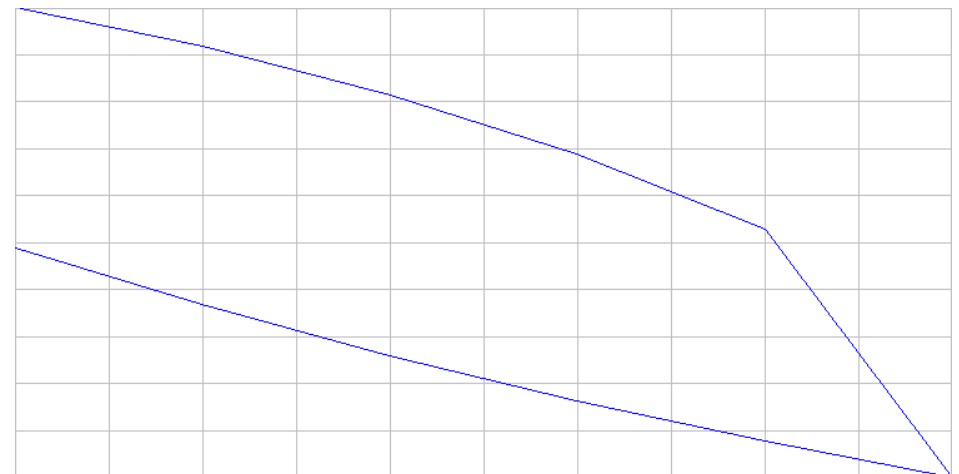
Pile Ultimate Capacity Analysis Results

Pile Data	
Pile Designation	16 In. Square
Pile Material	Concrete
Penetration of Pile into the Soil, ft.	60
Basic "diameter" or size of the pile, ft.	1.33333333333333
Cross-sectional Area of the Pile, ft ²	1.778
Pile Toe Area, ft ²	1.778
Perimeter of the Pile, ft.	5.333
Soil Data	
Type of Soil	SP
Specific Gravity of Solids	2.65
Void Ratio	0.51
Dry Unit Weight, pcf	109.5
Saturated Unit Weight, pcf	130.5
Soil Internal Friction Angle phi, degrees	32
Cohesion c, psf	0
SPT N ₆₀ , blows/ foot	20
CPT q _c , psf	211,600
Distance of Water Table from Soil Surface, ft.	40
Penetration of Pile into Water Table, ft.	20
Active Earth Pressure Coefficient (K _{min})	0.701
Frictional Angle Between Pile and Soil delta, degrees	27.9
Minimum Value for Beta	0.372

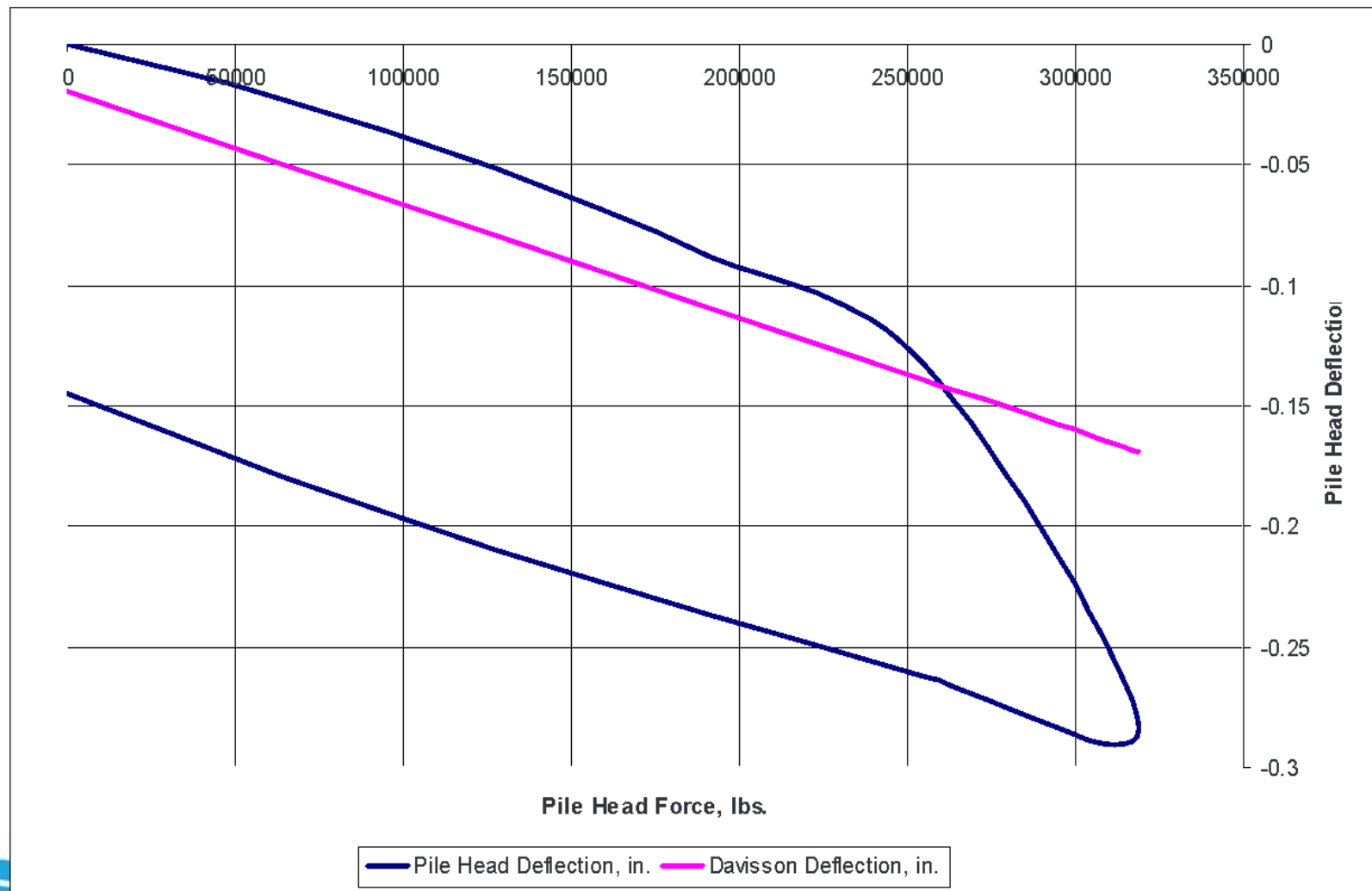
Pile Toe Results	
Effective Stress at Pile Toe, ksf	5.741
N _q	35.4
Relative Density at Pile Toe, Percent	45
SPT (N ₁) ₆₀ at pile toe, blows/ foot	12
Unit Toe Resistance q _p , ksf	203.4
Shear Modulus at Pile Toe, ksf	543.3
Toe Spring Constant Depth Factor	1.376
Toe Spring Constant, kips/ ft	3,392.2
Pile Toe Quake, in.	1.279
Poisson's Ratio at Pile Toe	0.310
Toe Damping, kips-sec/ ft	18.8
Toe Smith-Type Damping Constant, sec/ ft	0.052
Total Static Toe Resistance Q _p , kips	361.64
Pile Toe Plugged?	No
Final Results	
Total Shaft Friction Q _s , kips	276.16
Ultimate Axial Capacity of Pile, kips	637.80
Pile Setup Factor	1.0
Total Pile Soil Resistance to Driving (SRD), kips	637.80

● Settlement Example

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

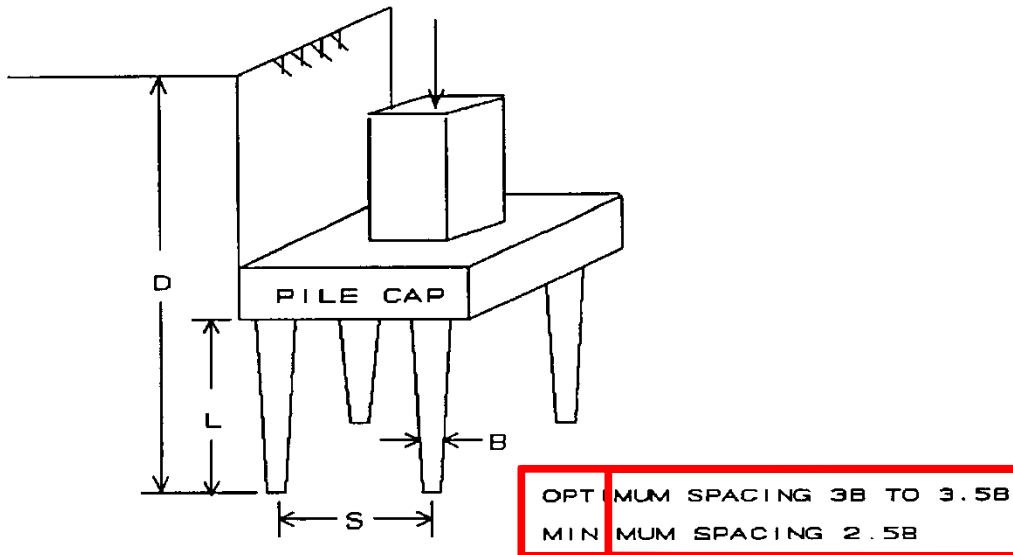


● Settlement Example

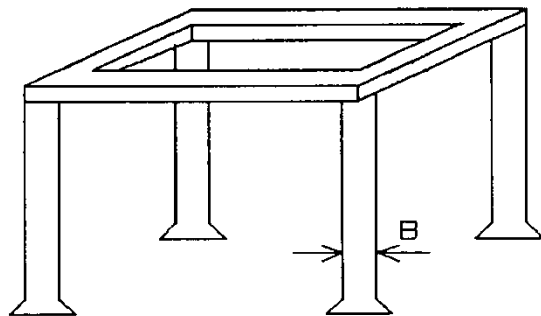


Davisson Capacity about 260 kips, at
intersection of lines

Group Effects



a. PILES



b. DRILLED SHAFTS

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

Stress Zones in Supporting Soils

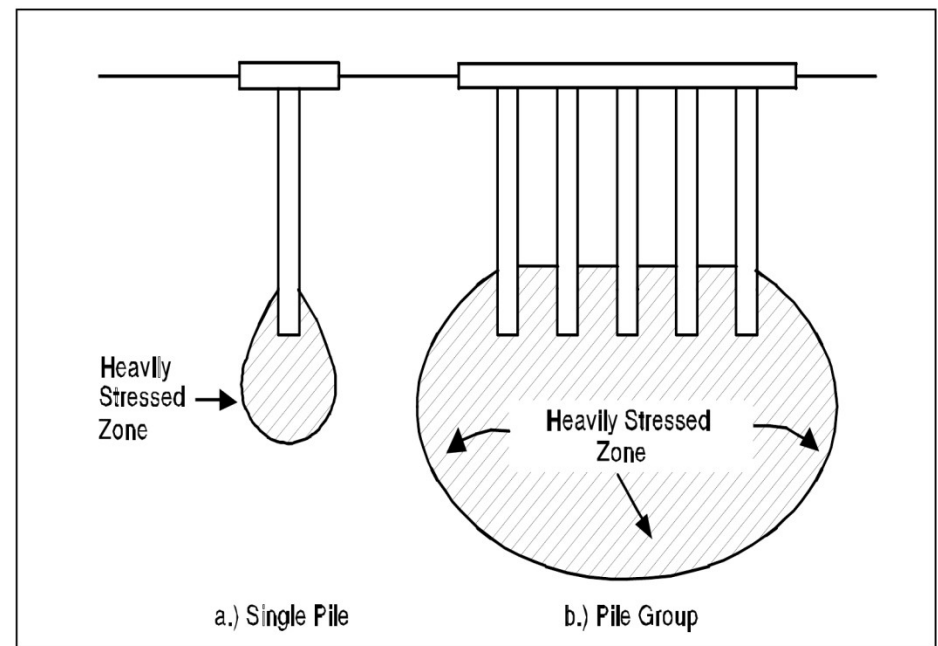


Figure 9-31. Stress zone from single pile and pile group (after Tomlinson, 1994).

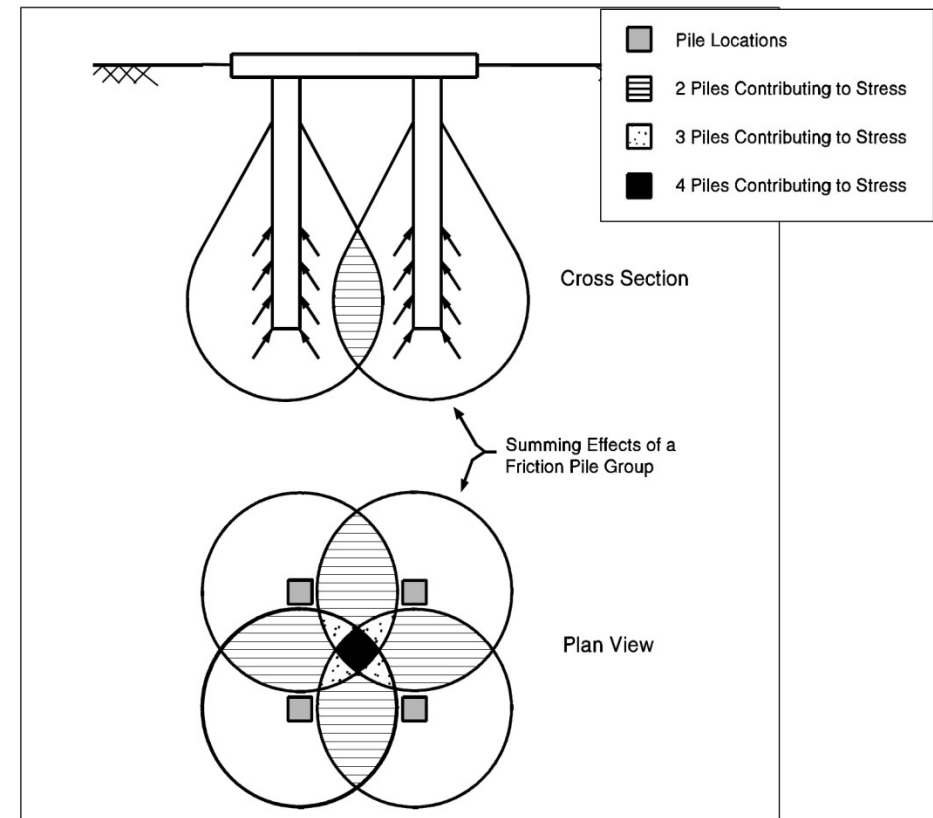


Figure 9-32. Overlap of stress zones for friction pile group (after Bowles, 1996).

9.6 DESIGN OF PILE GROUPS

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

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

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

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

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

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

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

9.6.1 Axial Compression Capacity of Pile Groups

9.6.1.1 Cohesionless Soils

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

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

1. The ultimate group capacity for driven piles in cohesionless soils not underlain by a weak deposit may be taken as the sum of the individual ultimate pile capacities, provided jetting or predrilling was not used in the pile installation process. Jetting or predrilling can result in group efficiencies less than 1. Therefore, jetting or predrilling should be avoided whenever possible or controlled by detailed specifications when necessary.
1. If a pile group founded in a firm bearing stratum of limited thickness is underlain by a weak deposit, then the ultimate group capacity is the smaller value of either the sum of the ultimate capacities of the individual piles, or the group capacity against block failure of an equivalent pier, consisting of the pile group and enclosed soil mass punching through the firm stratum into the underlying weak soil. From a practical standpoint, block failure in cohesionless soils can only occur when the center to center spacing of the piles is less than 2 pile diameters, which is less than the minimum center to center spacing of 2.5 diameters allowed by the AASHTO code (2002). The method shown for cohesive soils presented in the Section 9.6.1.3 may be used to evaluate the possibility of a block failure.
3. Piles in groups should not be installed at center to center spacings less than 3 times the average pile diameter. A minimum center to center spacing of 3 diameters is recommended to optimize group capacity and minimize installation problems.

9.6.1.2 Cohesive Soils

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

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

1. For pile groups driven in clays with undrained shear strengths of less than 2 ksf (95 kPa) and for the pile cap not in firm contact with the ground, a group efficiency of 0.7 should be used for center to center pile spacings of 3 times the average pile diameter. If the center to center pile spacing is greater than 6 times the average pile diameter, then a group efficiency of 1.0 may be used. Linear interpolation should be used for intermediate center to center pile spacings.
2. For pile groups driven in clays with undrained shear strengths less than 2 ksf (95 kPa) and for the pile cap in firm contact with the ground, a group efficiency of 1.0 may be used.
3. For pile groups driven in clays with undrained shear strength in excess of 2 ksf (95 kPa), a group efficiency of 1.0 may be used regardless of the pile cap - ground contact.
4. Calculate the ultimate pile group capacity against block failure by using the procedure described in Section 9.6.1.3.
5. Piles in groups should not be installed at center to center spacings less than 3 times the average pile diameter and not less than 3 ft (1 m).

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

9.6.1.3 Block Failure of Pile Groups

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

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

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

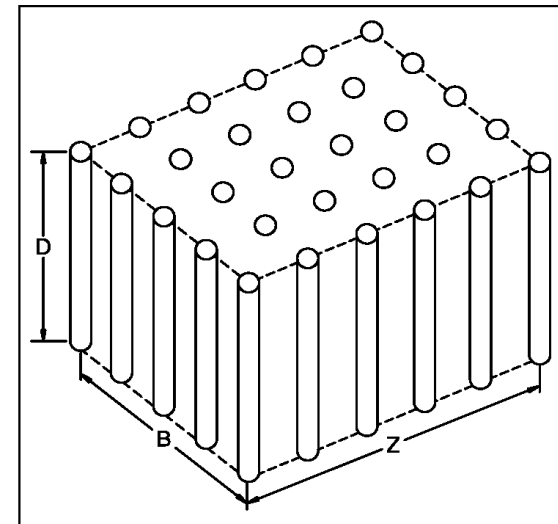


Figure 9-34. Three dimensional pile group configuration (after Tomlinson, 1994).

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

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

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

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

9.6.2 Settlement of Pile Groups

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

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

9.6.2.1 Elastic Compression of Piles

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

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

where: Δ = elastic compression of pile material in inches (mm)
 Q_a = design axial load in pile in kips (kN)
 L = length of pile in inches (mm)
 A = pile cross sectional area in in² (mm²)
 E = modulus of elasticity of pile material in ksi (kPa)

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

9.6.2.2 Settlement of Pile Groups in Cohesionless Soils

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

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

where: s = estimated total settlement in inches
 p_f = design foundation pressure in ksf = group design load divided by group area
 B = width of pile group in ft
 $\overline{N}'$ = average corrected SPT N_{160} value within a depth B below pile toe
 I_f = influence factor for group embedment = $1 - [D / 8B] \geq 0.5$
 D = pile embedment depth in ft

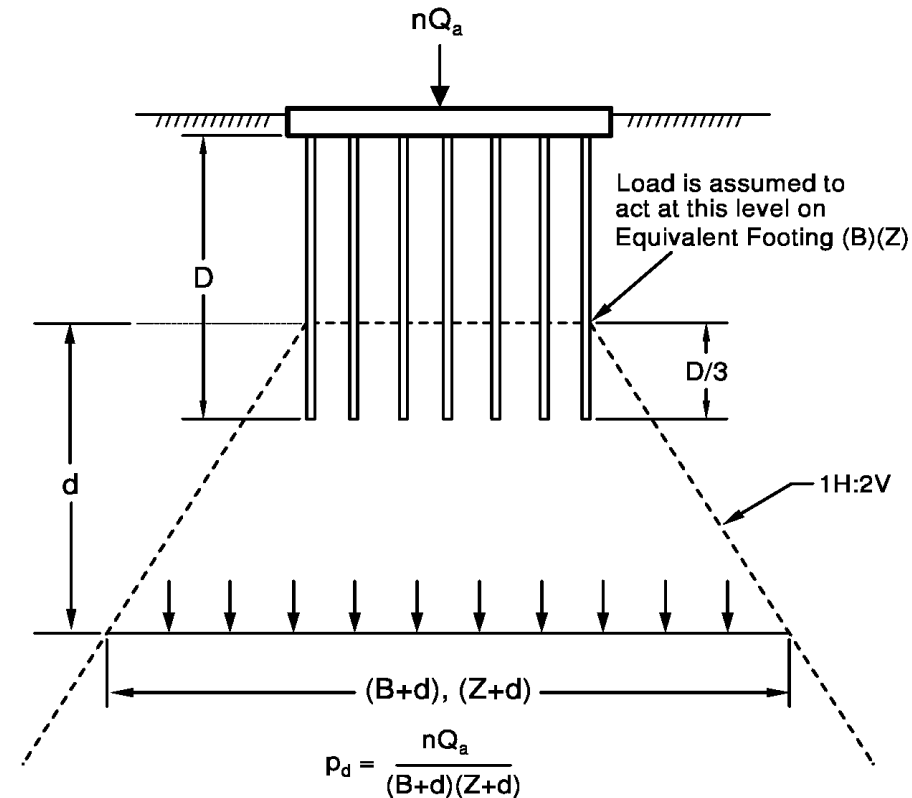
9.6.2.3 Settlement of Pile Groups in Cohesive Soils

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

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

- toe bearing piles in hard clay or sand underlain by soft clay
- piles supported by shaft resistance in clay
- piles supported in shaft resistance in sand underlain by clay
- piles supported by shaft and toe resistance in layered soil profile

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



Note: Pile Group has Plan Dimension of B and Z

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

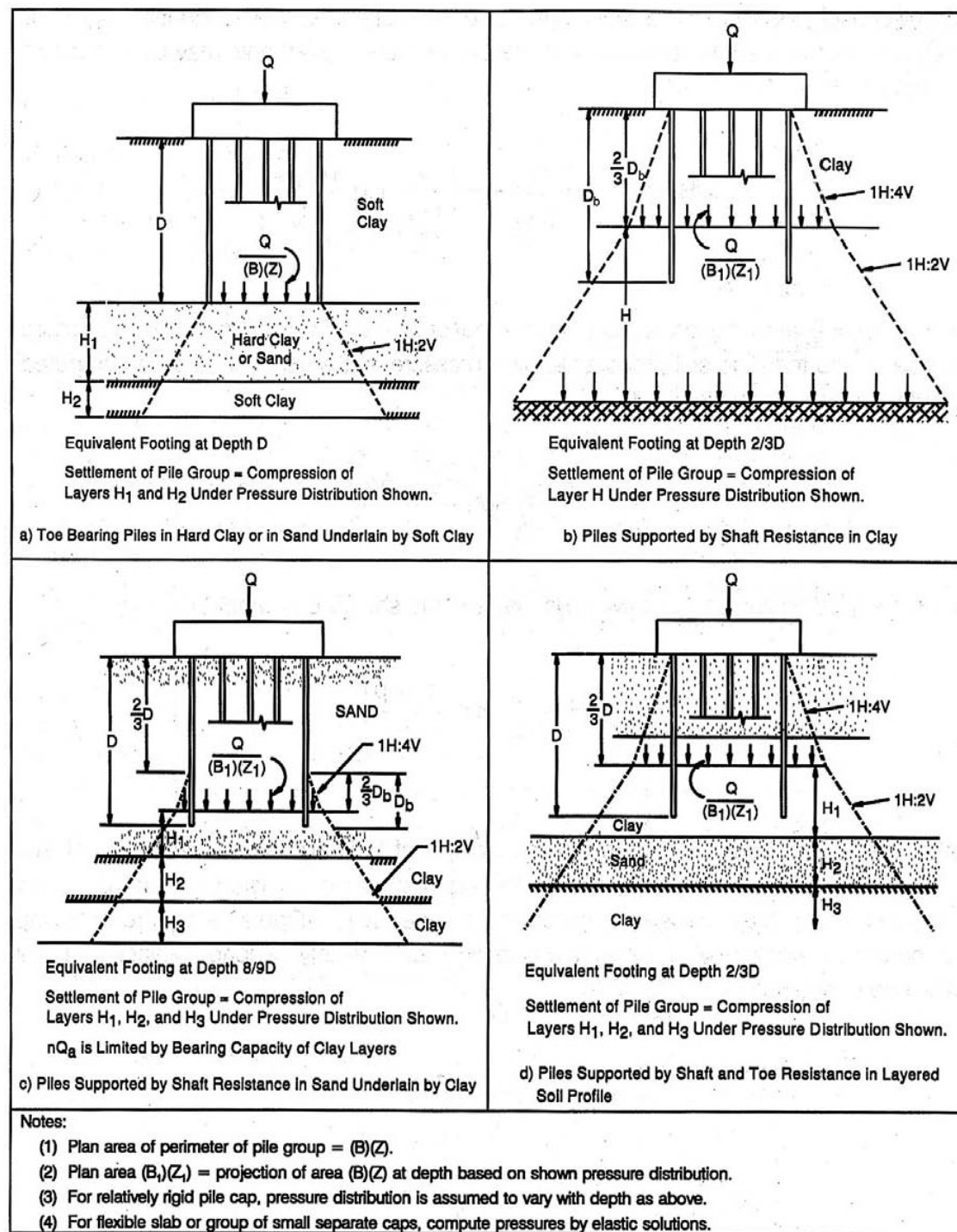


Figure 9-36. Stress distribution below equivalent footing for pile group (FHWA, 2006a).

● Group Settlement Example

- Pile Group

- 12” Square Concrete Piles, 50’ long/embedment
- Pile Cap 12’ x 12’ (B)
- Piles arranged in a 5 x 5 arrangement (25 piles total)
- Total cap load 1250 kips
- Group driven into SW soils, typical N_{160} value of 20

- Find

- Estimated group settlement

- Solution

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

$$s = \frac{4 p_f \sqrt{B} I_f}{\bar{N}'}$$

For silty sand, use: $s = \frac{8 p_f \sqrt{B} I_f}{\bar{N}'}$

9-21

where:

- s = estimated total settlement in inches
- p_f = design foundation pressure in ksf = group design load divided by group area
- B = width of pile group in ft
- $\bar{N}'$ = average corrected SPT N_{160} value within a depth B below pile toe
- I_f = influence factor for group embedment = $1 - [D / 8B] \geq 0.5$
- D = pile embedment depth in ft

Downdrag in Piles

When piles are installed through a soil deposit undergoing consolidation, the resulting relative downward movement of the soil around piles induces "**downdrag**" forces on the piles. These "downdrag" force is also called negative shaft resistance. Negative shaft resistance is the reverse of the usual positive shaft resistance developed along the pile surface that allows the soil to support the applied axial load. The downdrag force increases the axial load on the pile and can be especially significant on long piles driven through compressible soils. Therefore, the potential for negative shaft resistance must be considered in pile design. Batter piles should be avoided in soil conditions where relatively large soil settlements are expected because of the additional bending forces imposed on the piles, which can result in pile deformation and damage.

Settlement computations should be performed to determine the amount of settlement the soil surrounding the piles is expected to undergo after the piles are installed. The amount of relative settlement between soil and pile that is necessary to mobilize negative shaft resistance is about 0.4 to 0.5 inches (10 to 12 mm). At that amount of movement, the maximum value of negative shaft resistance is equal to the soil-pile adhesion. The negative shaft resistance can not exceed this value because slip of the soil along the pile shaft occurs at this value. It is particularly important in the design of friction piles to determine the depth at which the pile will be unaffected by negative shaft resistance. Only below that depth can positive shaft resistance provide support to resist vertical loads.

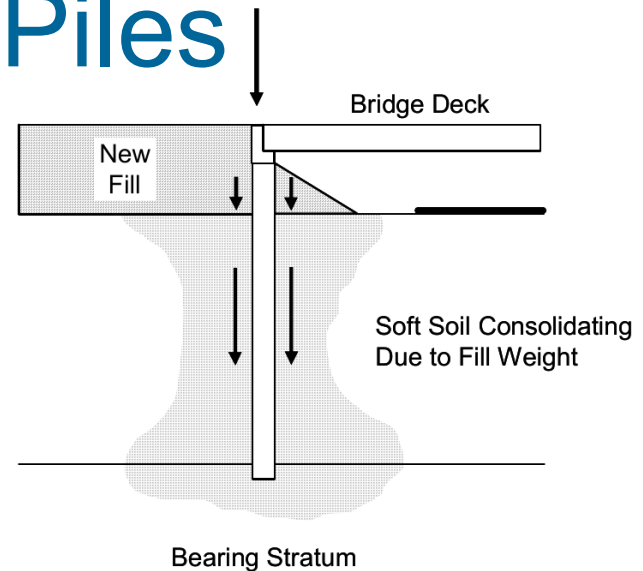


Figure 9-37(a). Common downdrag situation due to fill weight (FHWA, 2006a).

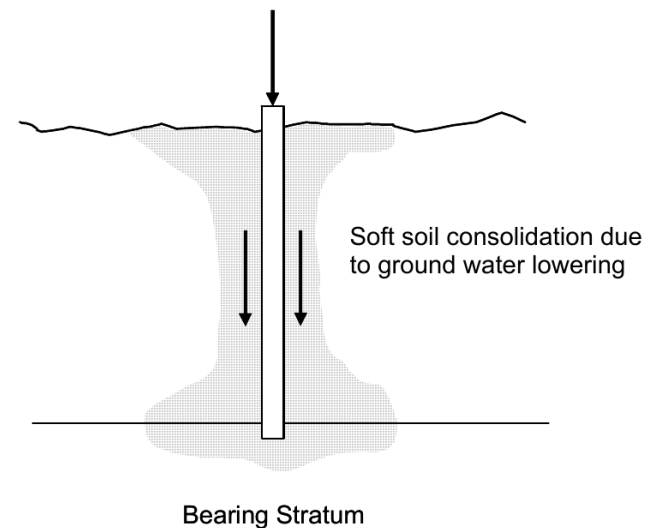


Figure 9-37(b). Common downdrag situation due to ground water lowering (FHWA, 2006a).

Questions

