An Introduction to the Design Methodology of FB-DEEP

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2012 Design Training Expo

Introduction

•FB-Deep stands for "Florida Bridge Deep Foundations";

•It is a Windows based program ;

•It can be used to analyze and estimate static axial capacity of either *driven piles* or *drilled shafts*

•It is updated and maintained by the Bridge Software Institute in the University of Florida

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Driven Piles

Driven pile designs use either:

- Standard Penetration Test (SPT), or
- Cone Penetration Test (CPT)

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Driven Piles - SPT

Background –
SPT Design Methodology Development

- 1967 - Dr. J. Schmertmann authored FDOT Research Bulletin No. 121-A titled "*Guideline for Use in the Soils Investigation and Design of Foundations for Bridge Structure in the Sate of Florida*"
- 1972 – L.C. Nottingham and R.H. Renfro coded a computer program SPT – FDOT Research Bulletin No. 121-B titled "*A Computer Program to Estimate Pile Load Capacity from Standard Penetration Test Results*". The code was written in Fortran based on pile foundation design methodology RB No. 121-A. SPT (mainframe)

Driven Piles – SPT (continue)

Background - SPT Design Methodology Development

- 1986 – Converted the main frame SPT to PC program and do multi-pile analyses in one single run by J. A. Caliendo, SPT (PC)
- 1989 - Revised SPT program based on pile load test database established in a FDOT funded Research Projects by McVay, Townsend, et al of University of Florida in 1987, SPT89
- 1991 – FDOT Structures Design Office rewrote the SPT89 code to make it more efficient and became SPT91
- 1994 – revised steel pile design based on Drs. McVay and Townsend's research, 1994; and add SI units by Lai, SPT94
- 1997 - Rewrote by FDOT Structures Design Office using C language to change the pre & post processors, SPT97
- 2004 – BSI expand SPT97 to include CPT pile design and combine SHAFT98 to FB-Deep

FB-DEEP Driven Piles –SPT

Basic Design Methodology

- Basic Design Methodology – Schmertmann's RB-121 A;
  - Empirically correlate static cone sounding and SPT N-values for both side and tip resistance of piles;
  - End bearing capacity – Account for soils 3.5B below and 8.0B above the pile tip (to safeguard against punching failure);
  - Ultimate side friction resistance - soil layers above the bearing layer and *in* the bearing layer are determined separately. A weighted average technique for side resistance is used to establish the ultimate unit skin friction in each layer;
  - Critical depth/pile width ratio corrections.

FB-DEEP Driven Piles –SPT

Basic Design Methodology

Empirically correlate static cone sounding and SPT N-values for both side and tip resistance of piles (original RB 121A values);

Type of Soil	USCS	qc/N	Fr (%)	Side Friction (tsf)	End Bearing (tsf)
Plastic Clay	CH, OH	1.0	5.0	0.05N	0.7N
Clay-Silt-Sand mixes, very silty sand; silts and marls	GC SC ML CL	2.0	2.0	0.04N	1.6N
Clean sands	GW, GP, GM, SW, SP, SM	3.5	0.6	0.019N	3.2N
Soft Limestones, Very shelly sands		4.0	0.25	0.01N	3.6N

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FB-DEEP Driven Piles –SPT

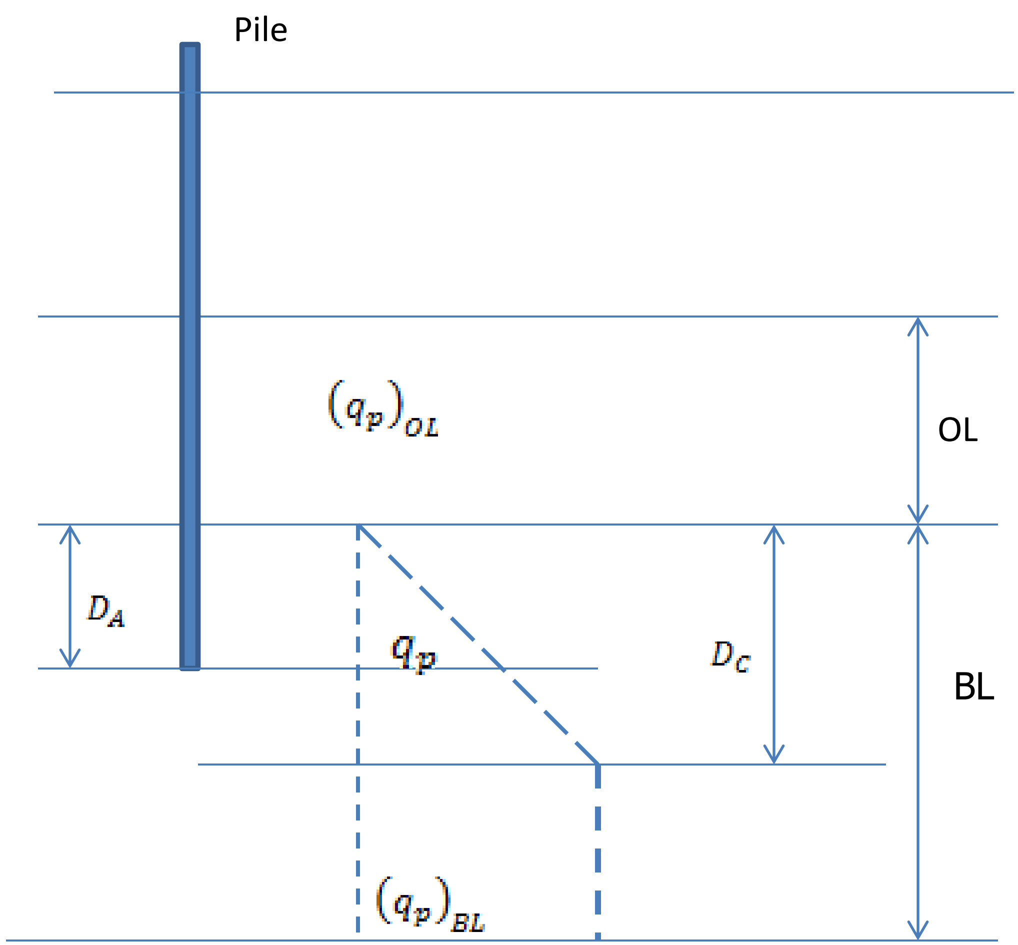
Basic Design Methodology

CRITICAL DEPTH CONCEPT AND CORRECTIONS

•Ultimate pile bearing capacity increase with the increase of embedment depth (D) in a soil layer until it reaches a depth-to-pile width/diameter (B) ratio at which the ultimate bearing capacity remains constant in the soil layer .

•The changes between the top of the soil layer and the critical depth embedment is considered linear,

•Ultimate bearing capacity for pile embedded in the soil layer above the critical depth needed corrections



Pile
$(q_p)_{OL}$
OL
D_A
q_p
D_C
BL
$(q_p)_{BL}$

FB-DEEP Driven Piles –SPT

Basic Design Methodology

End bearing resistance – Account for soils 3.5B below and 8.0B above the pile tip (to guard against punching failure);

- Bearing layer is overlain by a weak layer – correction should be made using the following equation:

$$q_p = (q_p)_{OL} + \frac{D_A}{D_C}\left[(q_p)_{BL} - (q_p)_{OL}\right]$$

where q_p = corrected unit pile tip bearing resistance

$(q_p)_{OL}$ = unit end bearing of layer above

$(q_p)_{BL}$ = unit end bearing of pile tip in the bearing layer

D_A = depth of pile embedment in the bearing layer

D_C = critical depth of embedment

- Bearing layer is overlain by a stronger layer – no correction

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Basic Design Methodology

CRITICAL DEPTH CONCEPT AND CORRECTIONS

Soil Type		Critical Depth Ratio (D/B)
1	Plastic Clay	2
2	Clay, Silty Sand	4
3	Clean Sand (N <= 12) (N <= 30) (N > 30)	 6 9 12
4	Limestone, Very Shelly Sand	6

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Basic Design Methodology

- Ultimate side friction resistance - soil layers above the bearing layer and *in* the bearing layer are determined separately. A weighted average technique for side resistance is used to establish the ultimate unit skin friction in each layer;

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Basic Design Methodology

CRITICAL DEPTH CORRECTIONS FOR END BEARING

If actual depth of embedment < critical depth, and when the bearing layer is stronger than the overlying layer, a correction (reduction) is applied to the unit end bearing capacity, by interpolating between the bearing capacity at the top of the bearing layer and the bearing capacity at the pile tip, as follows:

$$q = q_{LC} + \frac{D_A}{D_C}(q_T - q_{LC})$$

q = Corrected unit end bearing @ pile tip
q_{LC} = Unit end bearing at layer change
q_T = Uncorrected unit end bearing at pile tip
D_A = Actual embedment in bearing layer
D_C = Critical depth of embedment

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Basic Design Methodology

CRITICAL DEPTH CORRECTIONS FOR SIDE FRICTION

Pile tip embedment in the bearing layer is less than the critical depth and the overlying layer is weaker than the bearing layer, the side friction in the bearing layer is corrected (reduced) as follows:

$$CSFBL = \frac{SFBL}{q_T}\left[q_{LC} + \frac{D_A}{2D_C}(q_T - q_{LC}) \right]$$

CSFBL = Corrected side friction in the bearing layer

SFBL = Uncorrected side friction in the bearing layer

q_{LC} = Unit end bearing at layer change

q_T = Uncorrected unit end bearing at pile tip

D_A = Actual embedment in bearing layer

D_C = Critical depth of embedment

FB-DEEP Driven Piles –SPT

Basic Design Methodology

CRITICAL DEPTH CORRECTIONS FOR SIDE FRICTION

Pile tip embedment in the bearing layer is > the critical depth and the overlying layer is weaker than the bearing layer, the skin friction between the top of the bearing layer and the critical depth is corrected (reduced) as follows:

$$CSFACD = \frac{USFACD}{q_{CD}}\left[q_{LC} + 0.5(q_{CD} - q_{LC})\right]$$

CSFACD = Corrected side friction from top of bearing layer to the critical depth

USFACD = Uncorrected side friction from top of bearing layer to critical depth

q_{CD} = Unit end bearing at critical depth

q_{LC} = Unit end bearing at layer change

Driven Piles - SPT

Capacity Calculations

Ultimate Unit Side Friction

For Concrete Piles – square, round & cylinder with diameter ≤ 36”

Soil Type	*Ultimate Unit Side Friction (in TSF)*
1 – Plastic Clay	f = 2.0N (110 – N) / 4000.6
2 – Clay, Silty Sand	f = 2.0N (110 – N) / 4583.3
3 – Clean Sand	f = 0.019N
4 – Limestone, Very Shelly Sand	f = 0.01N

Driven Piles - SPT

Capacity Calculations

Mobilized Unit End Bearing

For Concrete Piles – square, round & cylinder with diameter ≤ 36”

Soil Type	Mobilized Unit End Bearing *(Tsf)*
1 – Plastic Clay	q = 0.7 * (N / 3)
2 – Clay, Silty Sand	q = 1.6 * (N / 3)
3 – Clean Sand	q = 3.2 * (N / 3)
4 – Limestone, Very Shelly Sand	q = 3.6 * (N / 3)

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Driven Piles - SPT

Capacity Calculations

Mobilized Unit End Bearing

For Concrete cylinder Piles with diameter > 36”

Soil Type	Mobilized Unit End Bearing *(Tsf)*
1 – Plastic Clay	*q = 0.2226 * (N / 3)*
2 – Clay, Silty Sand	q = 0.410 * (N / 3)
3 – Clean Sand	*q = 0.5676 * (N / 3)*
4 – Limestone, Very Shelly Sand	q = 3.6 * (N / 3)

Driven Piles - SPT
Capacity Calculations
Ultimate Unit Side Friction
Steel Pipe Piles (diameter ≤ 36”)

Soil Type	Ultimate Unit Side Friction *(in TSF)*
1 – Plastic Clay	$f_s = -8.081E{-4} + 0.058 * N - 1.202E{-3} * N^2 + 8.785E{-6} * N^3$
2 – Clay, Silty Sand	$f_s = 0.029 + 0.045 * N - 8.98E{-4} * N^2 + 6.371E{-6} * N^3$
3 – Clean Sand	$f_s = -0.026 + 0.023 * N - 1.435E{-4} * N^2 - 6.527E{-7} * N^3$
4 – Limestone, Very Shelly Sand	$f_s = 0.01 * N$

Driven Piles - SPT

Capacity Calculations

Mobilized Unit End Bearing

for steel pipe Piles (diameter ≤ 36")

Soil Type	Mobilized Unit End Bearing *(Tsf)*
1 – Plastic Clay	q = 0.7N / 3
2 – Clay, Silty Sand	q = 1.6N / 3
3 – Clean Sand	q = 3.2N / 3 for N≤30; q = [32 + 4(N – 30)]/30 for N>30
4 – Limestone, Very Shelly Sand	q = 1.2N for N≤30; q = [36 + 7(N – 30)]/30 for N>30

Driven Piles - SPT

Capacity Calculations

Ultimate Unit Side Friction

Steel Pipe Piles (diameter > 36”)

Soil Type	Ultimate Unit Side Friction *(in TSF)*
1 – Plastic Clay	*f_s = 0.4236*ln(N) – 0.5404*
2 – Clay, Silty Sand	f_s = 0.401 ln(N) – 0.463
3 – Clean Sand	*f_s = 0.2028*ln(N) -0.2646*
4 – Limestone, Very Shelly Sand	*f_s = 0.008 * N*

Based on the work of M.C. McVay, D. Badri, and Z.Hu, from the report "Determination of Axial Pile Capacity of Prestressed Concrete Cylinder Piles", 2004,

Driven Piles - SPT

Capacity Calculations

Mobilized Unit End Bearing

for steel pipe Piles (diameter > 36")

Soil Type	Mobilized Unit End Bearing* *(Tsf)*
1 – Plastic Clay	*q = 0.2226N*
2 – Clay, Silty Sand	*q = 0.4101N*
3 – Clean Sand	*q = 0.5676N*
4 – Limestone, Very Shelly Sand	q = 0.96N

*Based on the work of M.C. McVay, D. Badri, and Z.Hu, from the report "Determination of Axial Pile Capacity of Prestressed Concrete Cylinder Piles", 2004,

Driven Piles - SPT

Capacity Calculations

Ultimate Unit Side Friction

Steel H Piles

Soil Type	Ultimate Unit Side Friction *(in TSF)*
1 – Plastic Clay	$f = 2N(110 - N) / 5335.94$
2 – Clay, Silty Sand	$f = -0.0227 + 0.033N - 4.576E\text{-}4 * N^2 + 2.465E\text{-}6 * N^3$
3 – Clean Sand	$f = 0.00116N$
4 – Limestone, Very Shelly Sand	$f = 0.0076N$

Driven Piles - SPT

Capacity Calculations

Mobilized Unit End Bearing

Steel H Piles

Soil Type	Mobilized End Bearing (in TSF)
1 – Plastic Clay	$q_t = 0.7N / 3$
2 – Clay, Silty Sand	$q_t = 1.6N / 3$
3 – Clean Sand	$q_t = 3.2N / 3$
4 – Limestone, Very Shelly Sand	$q_t = 3.6N / 3$

Driven Piles – SPT

Data Input

Soil Type

Soil Type		Unified Soil Classifications
1	Plastic Clays	CH, OH
2	Clay-silt-sand mixes; Very silty sand; Silts and marls	GC, SC, ML, CL
3	Clean sands	GW, GP, GM, SW, SP, SM
4	Soft limestone; limerock; Very Shelly sands	
5	voids	

Driven Piles – SPT

Data Input

SPT N – value

- Safety hammer
- Un-corrected blow counts
- N-value ≤ 5 or ≥ 60 would be discarded in the calculations

Layering

- Split a thick soil layer into several sub-layers with similar N-values/relative density or consistency.
- Adjust the N-values for sub-soils that reveal shells base on local experience.
- Insert a thin dummy soil layer of different soil type between soil types or at soil layer breaks.

Entering Soil Data for Piles

ID	Depth (ft)	No. of Blows (Blows/ft)	Soil Type
1	0.00	0.00	3- Clean sand
2	1.00	0.00	3- Clean sand
3	3.00	0.00	3- Clean sand
4	6.00	0.00	3- Clean sand
5	9.00	0.00	3- Clean sand
6	12.00	0.00	3- Clean sand
7	15.00	5.00	3- Clean sand
8	18.00	7.00	3- Clean sand
9	21.00	7.00	3- Clean sand
10	[illegible]	7.00	2- Clay and silty sand
11	24.00	11.00	3- Clean sand
12	27.00	10.00	3- Clean sand
13	30.00	11.00	3- Clean sand
14	[illegible]	5.00	2- Clay and silty sand
15	33.00	5.00	3- Clean sand
16	36.00	4.00	3- Clean sand
17	39.00	3.00	3- Clean sand
18	42.00	6.00	3- Clean sand
19	45.00	5.00	3- Clean sand
20	48.00	4.00	3- Clean sand
21	52.00	5.00	3- Clean sand
22	[illegible]	5.00	2- Clay and silty sand
23	55.00	9.00	3- Clean sand
24	58.00	7.00	3- Clean sand
25	61.00	7.00	3- Clean sand
26	[illegible]	7.00	2- Clay and silty sand
27	64.00	44.00	3- Clean sand
28	67.00	39.00	3- Clean sand
29	[illegible]	16.00	2- Clay and silty sand
30	70.00	16.00	3- Clean sand
31	73.00	17.00	3- Clean sand
32	[illegible]	12.00	3- Clean sand
33	76.10	5.00	2- Clay and silty sand
34	79.00	5.00	3- Clean sand

Blowcount Average Per Soil Layer

Layer Num.	Starting Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	Average Blowcount (Blows/ft)	Soil Type
1	15.72	-8.18	23.90	2.36	3-Clean Sand
2	-8.18	-8.28	0.10	7.00	2-Clay and Silty Sand
3	-8.28	-14.38	6.10	10.51	3-Clean Sand
4	-14.38	-17.28	2.90	5.00	2-Clay and Silty Sand
5	-17.28	-39.18	21.90	4.54	3-Clean Sand
6	-39.18	-39.28	0.10	5.00	2-Clay and Silty Sand
7	-39.28	-48.18	8.90	7.67	3-Clean Sand
8	-48.18	-48.28	0.10	7.00	2-Clay and Silty Sand
9	-48.28	-51.38	3.10	43.84	3-Clean Sand
10	-51.38	-54.28	2.90	16.00	2-Clay and Silty Sand
11	-54.28	-60.38	6.10	16.43	3-Clean Sand
12	-60.38	-63.28	2.90	5.00	2-Clay and Silty Sand
13	-63.28	-69.18	5.90	5.00	3-Clean Sand
14	-69.18	-69.28	0.10	5.00	2-Clay and Silty Sand
15	-69.28	-78.28	9.00	9.67	3-Clean Sand
16	-78.28	-84.28	6.00	12.50	2-Clay and Silty Sand
17	-84.28	-102.28	18.00	98.00	4-Limestone, Very Shelly Sand

Driven Piles

CPT Design Methodology

There are three design methods included in the FB-Deep:

- Schmertmann – "Guidelines for Cone Penetration Test Performance and Design", 1978, FHWA-TS-78-209
- University of Florida – FDOT research project by Bloomquist, McVay and Hu, 2007.
- LCPC (Laboratoire Central des Ponts et Chaussées) - the French Highway Department by Bustamante and Gianeselli, 1982.

Driven Piles

CPT Design Methodology

Schmertmann's Method

•uses both cone tip resistance and sleeve friction to estimate pile resistance;

•Calculate average cone tip resistance by using minimum path rule.

Driven Piles

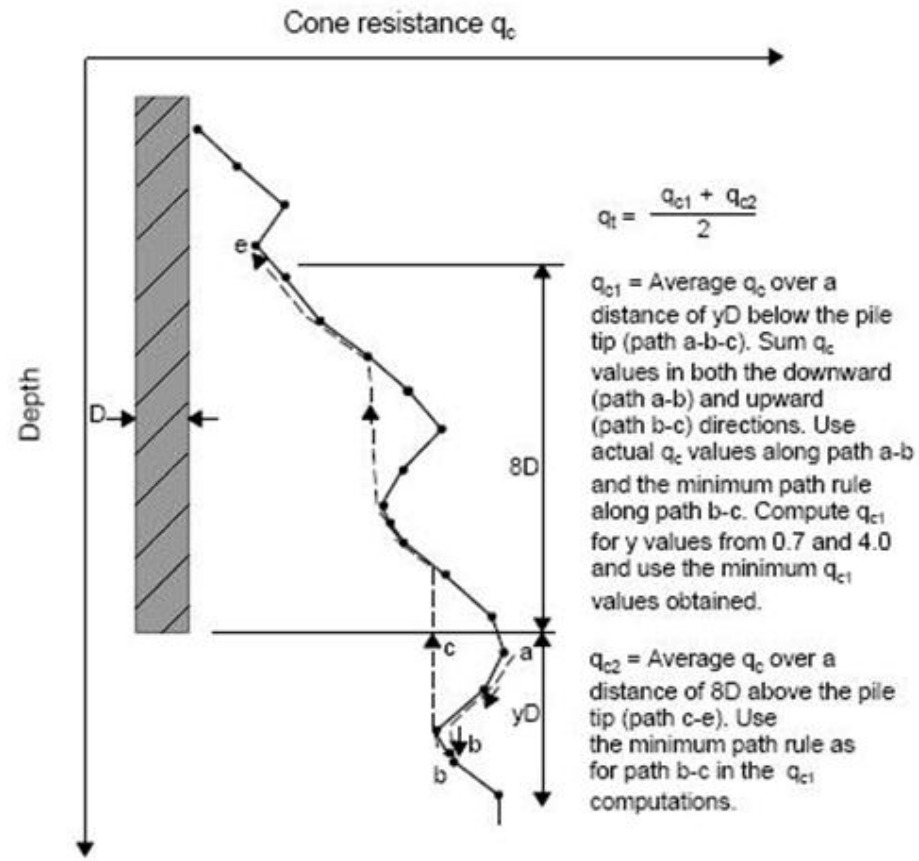
CPT Design Methodology

Schmertmann's Method

•Tip resistance - minimum path rule

- Consider cone resistances, q_c , between a depth of 8D above and yD below the pile tip
- Locate y below pile tip over a range of 0.7D and 4D and calculate the average q_{c1} as well as q_{c2} using min. path rule,
- Calculate total tip resistance:

$$q_t = (q_{c1} + q_{c2})/2$$



Driven Piles

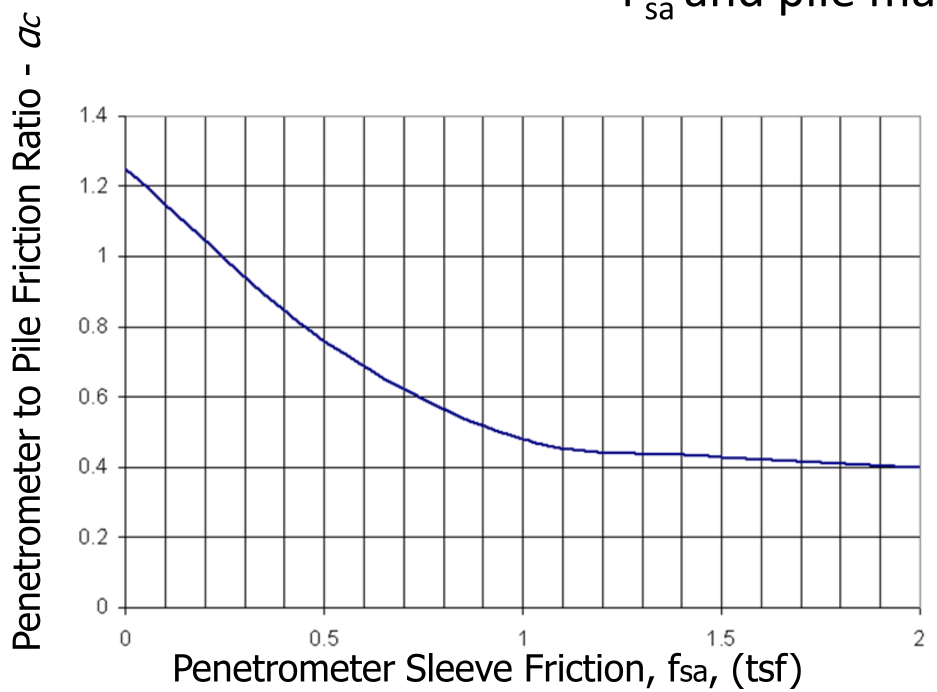
CPT Design Methodology

Schmertmann's Method

Concrete pile - Calculate side resistance in Clay

$$f_s = \alpha_c f_{sa} \le 1.2(tsf)$$

where: α_c is a function of f_{sa} and pile material



Driven Piles

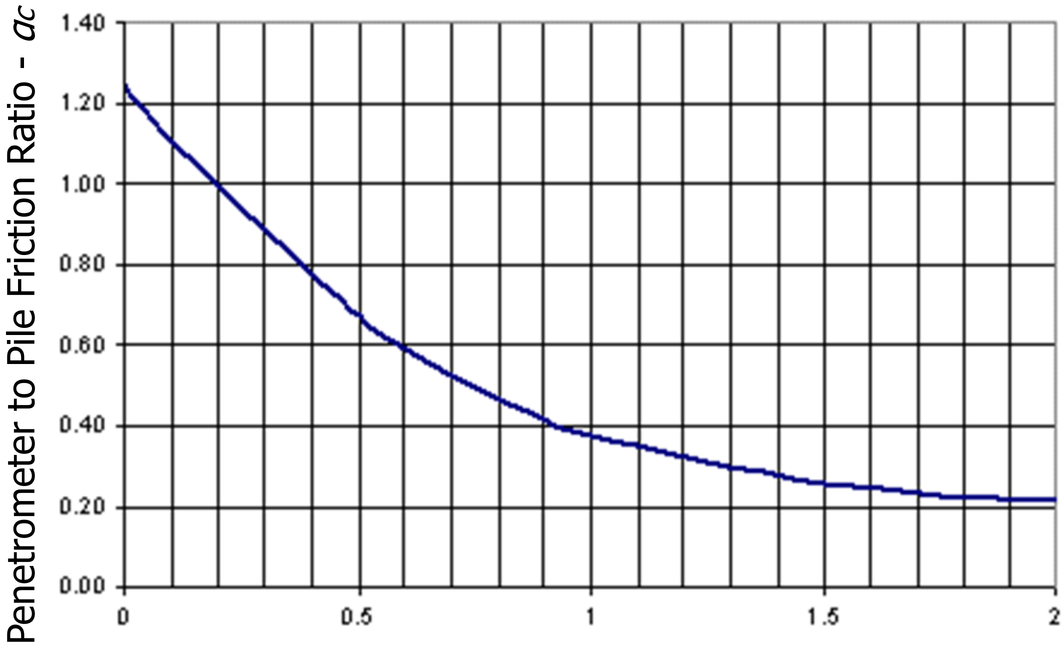
CPT Design Methodology

Schmertmann's Method

Steel pile - side friction in Clay

$$f_s = \alpha_c f_{sa} \leq 1.2tsf$$

where: α_c is a function of f_{sa} and pile material

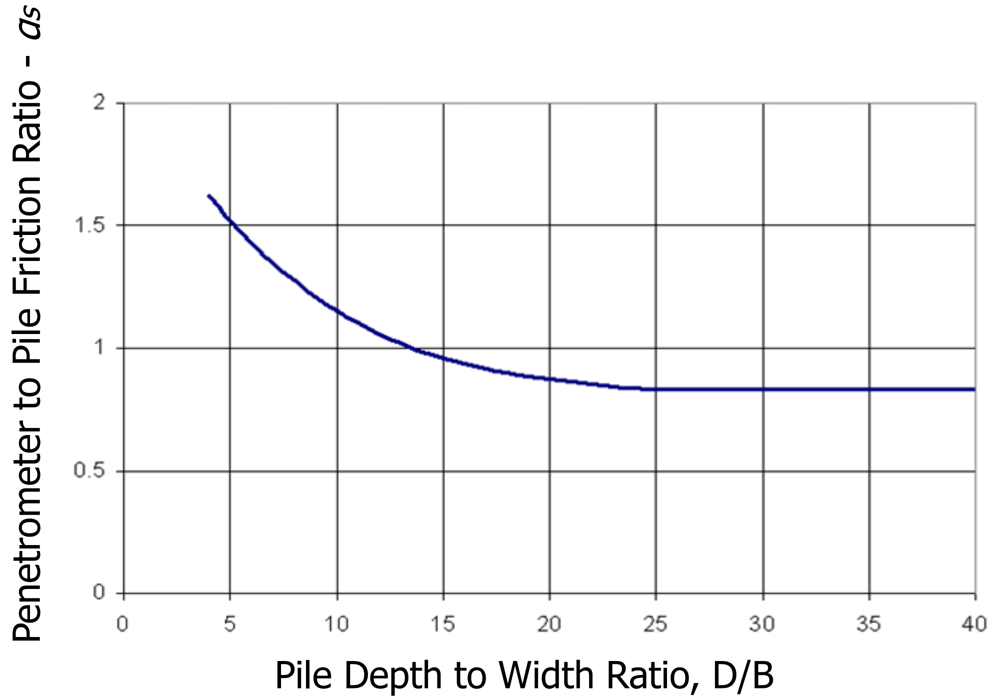


Penetrometer Sleeve Friction, f_{sa}, (tsf)

Driven Piles
CPT Design Methodology
Schmertmann's Method
Concrete pile - side friction in Sand

$$Q_s = \alpha_s \left(\sum_{y=0}^{8D} \frac{y}{8D} f_{sa} A_s + \sum_{y=8D}^{L} f_{sa} A_s \right)$$

where: α_s



Driven Piles

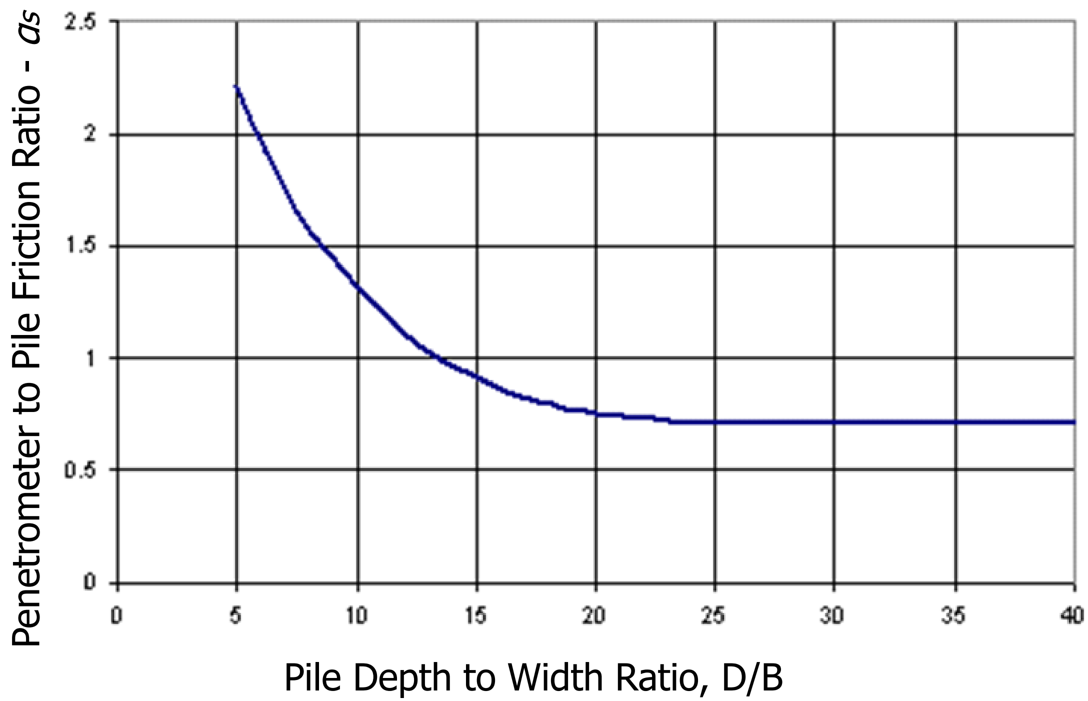
CPT Design Methodology

Schmertmann's Method

Steel pile - side friction in Sand

$$Q_s = \alpha_s \left(\sum_{y=0}^{8D} \frac{y}{8D} f_{sa} A_s + \sum_{y=8D}^{L} f_{sa} A_s \right)$$

where: α_s



Driven Piles

CPT Design Methodology

UF (university of Florida) Method

- soil type was determined by simplified soil classification chart for standard electronic friction cone (Robertson et al, 1986)
- using both CPT tip resistance and sleeve friction,
- Soil cementation was determined by SPT samples, DTP tip2/tip1 ratio or SPT qc/N ratio (>10)

Driven Piles

CPT Design Methodology

UF (university of Florida) Method

- Tip resistance

$$q_t = k_b * q_{ca}(\text{tip}) \leq 150 \text{ tsf}$$

Where $k_b =$

Well Cemented Sand	Lightly Cemented Sand	Gravel	Sand	Silt	Clay
0.1	0.15	0.35	0.4	0.45	1.0

$q_{ca}(\text{tip}) = (q_{ca\ above} + q_{ca\ below}) / 2$

$q_{ca\ above}$: average q_c measured from the tip to 8 D above the tip;

$q_{ca\ below}$: average q_c measured from the tip to 3 D below the tip for sand or 1D below the tip for clay

Driven Piles

CPT Design Methodology

UF (university of Florida) Method

- Side resistance from the CPT tip resistance, q_c

$$f_s = q_{ca}\,(\text{side}) * 1.25 / F_s \quad \leq 1.2 \text{ tsf}$$

where

Fs: friction factor that depends on the soil type as shown

Well Cemented Sand	Lightly Cemented Sand	Gravel and Dense Sand	Medium Dense Sand	Loose Sand	Silt, Sandy Clay, Clayey Sand	Clay
300	250	200	150	100	60	50

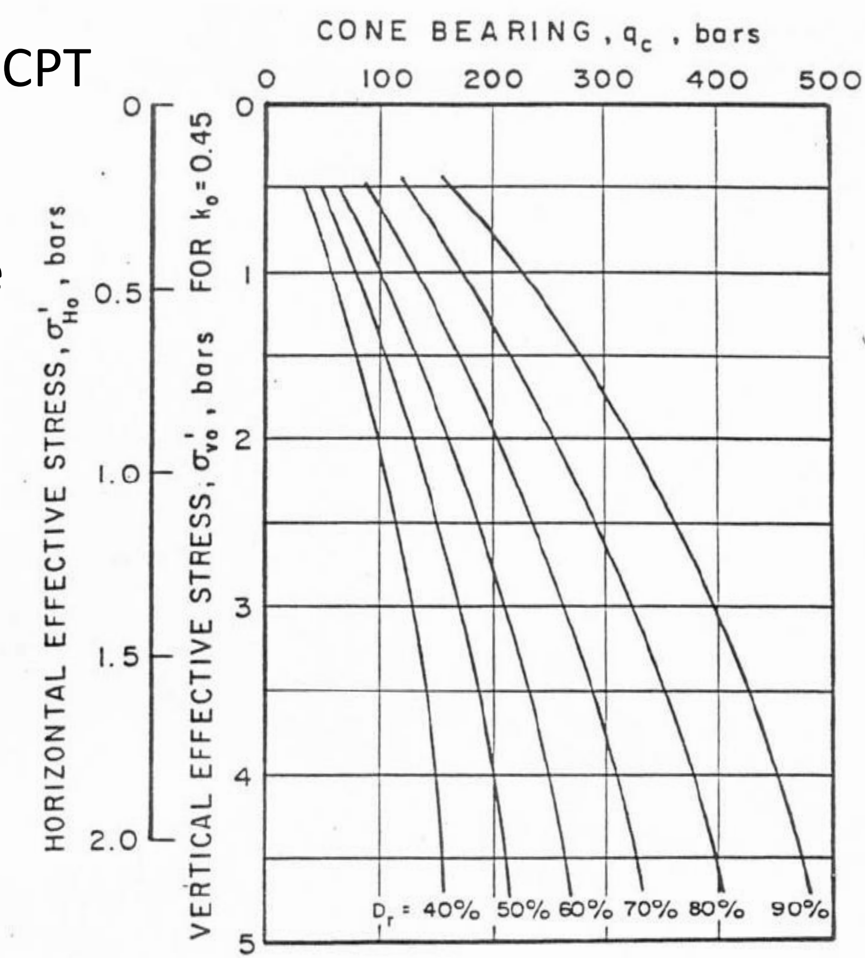
q_{ca} (side): the average q_c within the calculating soil layers along the pile

Driven Piles

CPT Design Methodology

UF (university of Florida) Method

- Side resistance from the CPT tip resistance, q_c
- Relative density can be obtained according to the chart to the right



Driven Piles

CPT Design Methodology

LCPC (or French) Method

- Uses only cone tip resistance for predicting axial pile capacity;
- Can be used for both driven piles and cast-in-place foundations (bored piles or drilled shafts)

Driven Piles

CPT Design Methodology

LCPC (or French) Method

Tip Resistance

$$q_t = k_b q_{eq}$$

where:

q_{eq} (tip) is the average cone tip resistance within 1.5 D above and 1.5 D below the pile tip after eliminating out of the range of ±30% of the average value, and

k_b is a cone bearing capacity factor based on pile installation procedure and soil type

Soil Type	Bored Piles	Driven Piles
Clay - Silt	0.375	0.600
Sand – Gravel	0.150	0.375
Chalk	0.200	0.400

Driven Piles

CPT Design Methodology

LCPC (or French) Method

Side Resistance

Select pile category:

Group I –

Pile type	Descriptions
1. FS drilled shaft with no drilling mud	Installed without supporting the soil with drilling mud. Applicable only for cohesive soils above the water table.
2. FB drilled shaft with drilling mud	Installed using mud to support the sides of the hole. Concrete is poured from the bottom up, displacing the mud.
3. FT drilled shaft with casting (FTU)	Drilled within the confinement of a steel casing. As the casing is retrieved, concrete is poured in the hole.
4. FTC drilled shaft, hollow auger (auger cast piles)	Installed using a hollow stem continuous auger having a length at least equal to the proposed pile length. The auger is extracted without turning while, simultaneously, concrete is injected through the auger stem.
5. FPU Pier	Hand excavated foundations. The drilling method requires the presence of workers at the bottom of the excavation. The sides are supported with retaining elements or casing.
6. FIG Micropile type I (BIG)	Drilled pile with casing. Diameter less than 250 mm (10 in). After the casing has been filled with concrete, the top of the casing is plugged. Pressure is applied inside the casing between the concrete and the plug. The casing is recovered by maintaining the pressure against the concrete.

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Driven Piles

CPT Design Methodology

LCPC (or French) Method

Side Resistance

Select pile category:

Group II

7. VMO screwed-in piles	Not applicable for cohesionless or soils below water table. A screw type tool is placed in front of a corrugated pipe which is pushed and screwed in place. The rotation is reversed for pulling out the casing while concrete is poured.
8. BE driven piles, concrete coated	- Pile piles 150 mm (6 in) to 500 mm (20 in) external diameter. - H piles. - Caissons made of 2, 3, or 4 sheet pile sections. The pile is driven with an oversized protecting shoe. As driving proceeds, concrete is injected through a hose near the oversized shoe producing a coating around the pile.
9. BBA driven prefabricated piles	Reinforced or prestressed concrete piles installed by driving or vibrodriving.
10. BM steel driven piles	Piles made of steel only and driven in place. - H piles. - Pipe piles. - Any shape obtained by welding sheet-pile sections.
11. BPR prestressed tube pile	Made of hollow cylinder elements of lightly reinforced concrete assembled together by prestressing before driving. Each element is generally 1.5 to 3 m (4-9 ft) long and 0.7 to 0.9 m (2-3 ft) in diameter. The thickness is approximately 0.15 m (6 in). The piles are driven open ended.
12. BFR driven pile, bottom concrete plug	Driving is achieved through the bottom concrete plug. The casing is pulled out while low slump concrete is compacted in it.

Driven Piles

CPT Design Methodology

LCPC (or French) Method

Side Resistance

Select pile category:

Group III

13. BMO driven piles, molded	A plugged tube is driven until the final position is reached. The tube is filled with medium slump concrete to the top and the tube is extracted.
14. VBA concrete piles, pushed-in	Pile is made of cylindrical concrete elements prefabricated or cast-in-place, 0.5 to 2.5 m (1.5 to 8 ft) long and 30 to 60 cm (1 to 2 ft) in diameter. The elements are pushed in by a hydraulic jack.
15. VME steel piles, pushed-in	Piles made of steel only are pushed in by a hydraulic jack.
16. FIP micropile type II	Drilled pile < 250 mm (10 in) in diameter. The reinforcing cage is placed in the hole and concrete placed from bottom up.
17. BIP high pressure injected pile, large diameter	Diameter > 250 mm (10 in). The injection system should be able to produce high pressures.

Driven Piles

CPT Design Methodology

LCPC(or French) Method

Side Resistance

- Select pile category based on pile installation procedure
- Determine soil design curve
    - Clay and Silt

Curve number	qc (ksf)	Pile type	Comments on insertion procedure
1	< 14.6 > 14.6	1-17 1, 2	- Very probable values when using tools without teeth or with oversized blades and where a remolded layer of material can be deposited along the sides of the drilled hole. Use these values also for deep holes below the water table where the hole must be cleaned several times. Use these values also for cases when the relaxation of the sides of the hole is allowed due to incidents slowing or stopping the pouring of concrete. For all the previous conditions, experience shows, however, that q_s can be between curve 1 and 2; use an intermediate value of q_s if such value is warranted by a load test.
2	> 25.1	4, 5, 8, 9, 10, 11, 13, 14, 15	- For all steel piles, experience shows that in plastic soils, q_s is often as low as curve 1; therefore, use curve 1 when no previous load test is available. For all driven concrete piles use curve 3 in low plasticity soils with sand or sand and gravel layers or containing boulders and when q_c > 52.2 ksf.
	> 25.1	7	- Use these values for soils where q_c < 52.2 ksf and the rate of penetration is slow; otherwise use curve 1. Also for slow penetration, when q_c > 93.9 ksf, use curve 3.
	> 25.1	6	- Use curve 3 based on previous load test.
	> 25.1	1, 2	- Use these values when careful method of drilling with an auger equipped with teeth and immediate concrete pouring is used. In the case of constant supervision with cleaning and grooving of the borehole walls followed by immediate concrete pouring, for soils of q_c > 93.9 ksf, curve 3 can be used.
	> 25.1	3	- For dry holes. It is recommended to vibrate the concrete after taking out the casing. In the case of work below the water table, where pumping is required and frequent movement of the casing is necessary, use curve 1 unless load test results are available.
3	> 25.1 < 41.8	12	- Usual conditions of execution as described in DTP 13.2
5	> 14.8	16, 17	- In the case of injection done selectively and repetitively at low flow rate it will be possible to use curve 5, if it is justified by previous load test.

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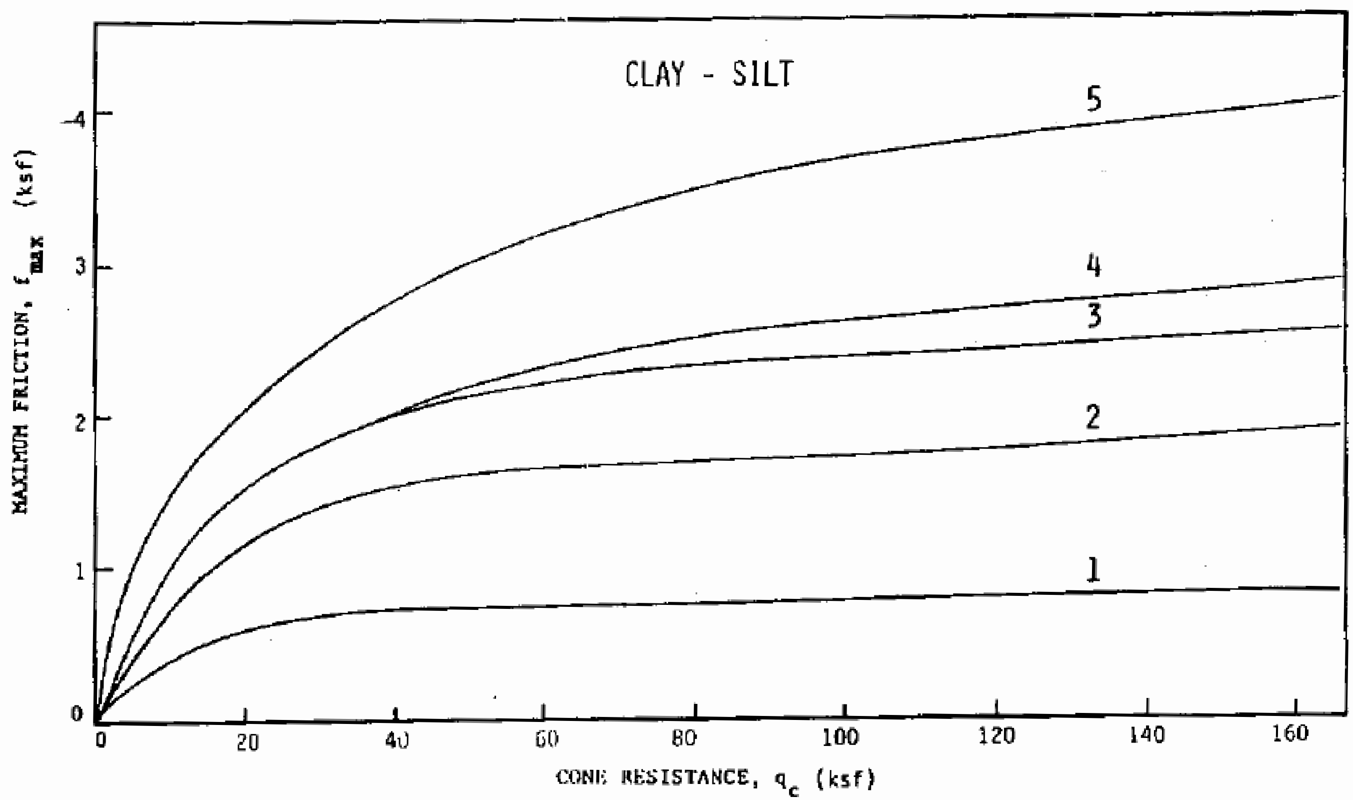
Driven Piles
CPT Design Methodology
LCPC (or French) Method



Driven Piles CPT Design Methodology

LCPC (or French) Method

Side Resistance

- Select pile category based on pile installation procedure
- Determine soil design curve
  - Sand and Gravel

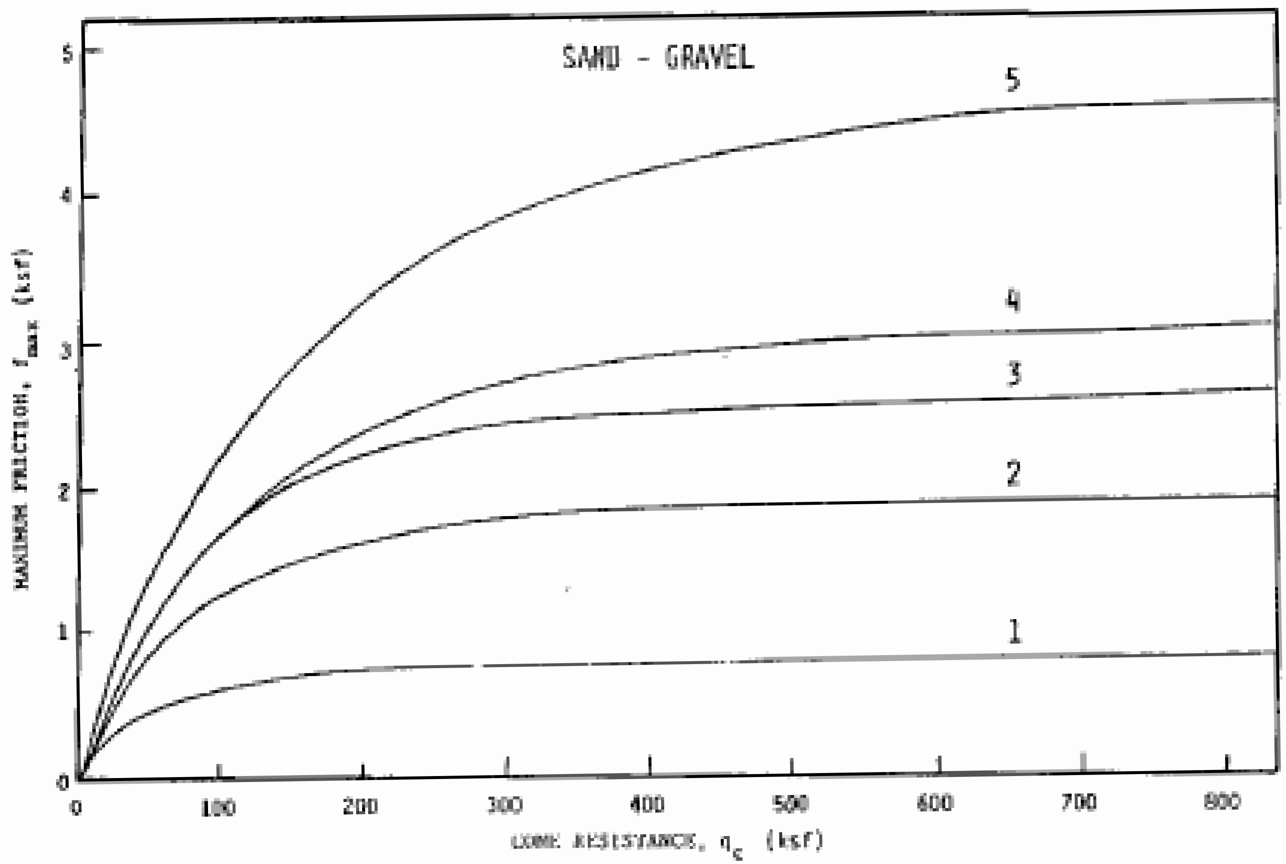
Table 2-9. Curve No. for sand and gravel from the LCPC Method

Curve number	qc (ksf)	Pile type	Comments on insertion procedure
1	< 73.1	2 - 4, 6 -15	
2	> 73.1	6, 7, 9 -15	- For fine sands. Since steel piles can lead to very small values of q_s in such soils, use curve 1 unless higher values can be based on load test results. For concrete piles, use curve 2 for fine sands of q_c > 156.6 ksf.
	> 104.4	2, 3	- Only for fine sands and bored piles which are less than 30m (100 ft) long. For piles longer than 30 m (100 ft) in fine sand, q_s may vary between curves 1 and 2. Where no load test data is available, use curve 1.
	> 104.4	4	- Reserved for sands exhibiting some cohesion.
3	> 156.6	6, 7, 9 - 11, 13 - 15, 17	- For coarse gravelly sand or gravel only. For concrete piles, use curve 4 if it can be justified by a load test.
	> 156.6	2, 3	- For coarse gravelly sand or gravel and bored piles less than 30 m (100 ft) long. - For gravel where q_c > 83.5 ksf, use curve 4.
4	> 156.6	8, 12	- For coarse gravelly sand and gravel only.
5	> 104.4	16, 17	- Use of values higher than curve 5 is acceptable if based on load test.

Driven Piles

CPT Design Methodology

LCPC (or French) Method



Drilled Shafts Design
Introduction

1. ShaftUF – a spread sheet program used FHWA Design Methods authored by Michael O'Neil and Lyman Reese, 1988 for sand & clay but without settlement calculation & user provide side friction for rock;
2. Shaft98 – Replace ShaftUF based on the works by Townsend et al. It's a Window base software based on FHWA Design Methods for sand, clay & Intermediate Geomaterials - FHWA Publication – IF-99-025 & McVay's Method for Florida Limestone;
3. FB-Deep – A modification of Shaft98, user can choose to input side friction for rock by either McVay's method or other correlations of q_u.

Drilled Shafts

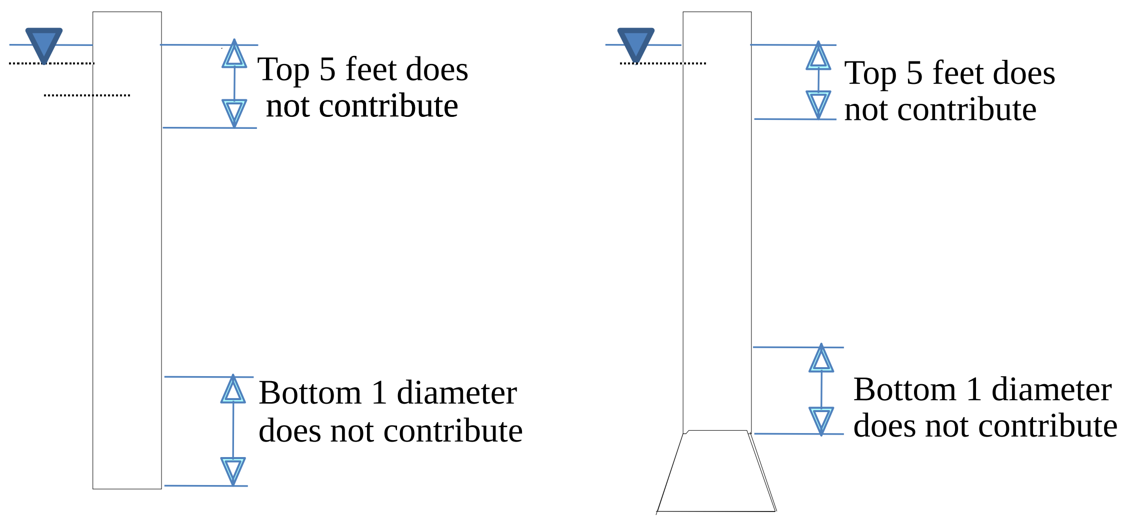
Axial Capacity

$$Q_t = \quad Q_s + Q_b$$

$$where \quad Q_t = \text{nominal shaft resistance}$$
$$Q_s = \text{ultimate side resistance}$$
$$Q_b = \text{nominal base resistance}$$

Drilled Shafts

Side Friction in Clay



Assumptions and Notes:

α = 0	for the top 5 feet of clay along the shaft.
α = 0	for the bottom 1 diameter width along the shaft.
α = 0	from the ground surface to the length of casing.
α = 0.55	along all other points of shaft

Drilled Shafts

Side Friction in Clay

$$f_{su} = \quad \alpha C_u$$

Where f_{su} = *ultimate side friction* ≤ 2.75 *tsf*

α = *empirical adhesion factor 0.55*

C_u = undrained shear strength

$$Q_s = \quad \int_{L_1}^{L_2} f_{su} dA$$

Where *dA = differential area of the perimeter along the shaft*

L_1 *&* L_2 *=penetration of drilled shaft between two soil layers*

Drilled Shafts

Base Resistance in Clay

$$q_b = N_c * C_u < 40\ tsf$$

where

q_b = unit base resistance for drilled shafts in clay
$N_c = 6.0[1 + 0.2(L/B)]$ $N_c < 9$
C_u = average undrained shear strength
L = total embedment length of shaft
B = diameter of shaft base.

- FB-Deep interpolates or extrapolates values of C_u at depths of one base diameter of the shaft below the base.
- FB-Deep takes a weighted average of C_u values between the base and three diameter widths below the base, where the shaft base is at the top of a clay layer.
- In those rare instances where the clay at the base is soft, the value of C_u may be reduced by one-third to account for local (high strain) bearing failure.

Drilled Shafts

Calculations for Base Resisance in Clay

- If drilled shafts with diameter >75 inches (1.9 m), tipped in stiff to hard clay, the q_b should be reduced to

$$q_{br} = F_r * q_b$$

where: $F_r = 2.5 / (aB_b + 2.5\,b)$ $F_r < 1$

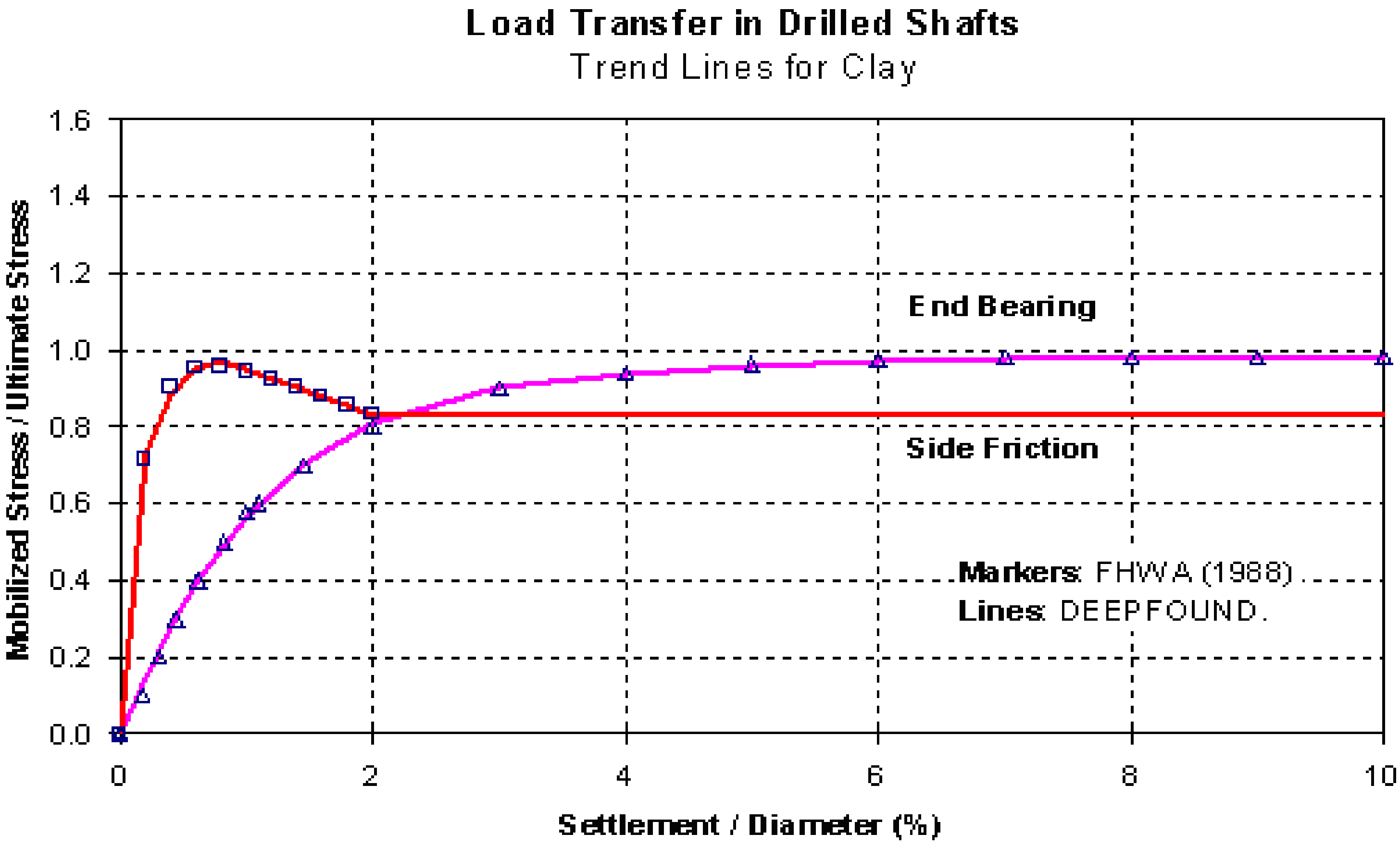
in which $a = 0.0071 + 0.0021\,(L/B_b)$, $a < 0.015$

$b = 0.45\,(C_{ub})^{0.5}$ $0.5 < b < 1.5$ and C_{ub} (ksf)

B_b = shaft diameter in inches

C_{ub}= average undrained shear strength of the soil between the base and $2B_b$ below the base.

Drilled Shafts
Settlement Trend Lines in Clay



Drilled Shaft

Short-term settlement (clay)

Alternate method

- Mobilized Side Friction

$f_s/f_{smax} = 0.593157*R/0.12$ for $R \leq 0.12$

$f_s/f_{smax} = R/(0.095155+0.892937*R)$ for $R \leq 0.74$

$f_s/f_{smax} = 0.978929 - 0.115817*(R-0.74)$ for $R \leq 2.0$

$f_s/f_{smax} = 0.833$ for $R > 2.0$

- Mobilized Base Resistance

$q_b/q_{bmax} = 1.1823E\text{-}4*R^5 - 3.709E\text{-}3*R^4 + 4.4944E\text{-}2*R^3 - 0.26537*R^2$
$+0.78436*R$ for $R \leq 6.5$

$q_b/q_{bmax} = 0.98$ for $R > 6.5$

$$R = \frac{S}{B} \times 100 \qquad S = settlement \qquad B = diameter\ of\ shaft$$

Drilled Shafts

Side Resistance in Clean Sand

$$f_{sz} = K \tan\phi_c \sigma_z = \beta \sigma_z$$

f_{sz} is ultimate unit side resistance in sand at depth z

σ_z is vertical effective stress at depth z

$$Q_s = \int \beta \sigma_z dA$$

dA is differential area of perimeter along sides of drilled shaft

$$\beta = 1.5 - 0.135\sqrt{z}$$

The value of β in the above equations is modified in certain cases, depending on depth and blowcount (see next slide)

Drilled Shafts

Calculations for Skin Resistance in Clean Sand (cont.)

Beta Values:

$$0.25 \leq \beta \leq 1.2$$

If the SPT N-Value is less than 15, β should be adjusted as follow:

$$\beta = (N/15)*\beta$$

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Drilled Shafts

Calculations for End Bearing in Clean Sand

For shafts less than 50 inches in diameter:

$$q_b = 0.6N_{60} \qquad N_{60} \leq 50$$

q_b is average unit end bearing

For shafts greater than 50 inches in diameter:

$$q_{br} = 50\left(\frac{q_b}{B_b}\right)$$

Weighted average N-values of 1.5B above and 2B below the shaft tip using the following equation for end bearing capacity calculation;

$$N_{spt} = \frac{\sum N_k L_k}{\sum L_k}$$

- L is thickness of Layer k; N_{spt} is blowcount for layer k

Drilled Shaft

Design for Sand

Mobilized Side Friction

f_s/f_{smax} = -2.16*R^4 + 6.34*R^3 - 7.36*R^2 + 4.15*R   for $R \leq 0.908333$

f_s/f_{smax} = 0.978112   for $R > 0.908333$

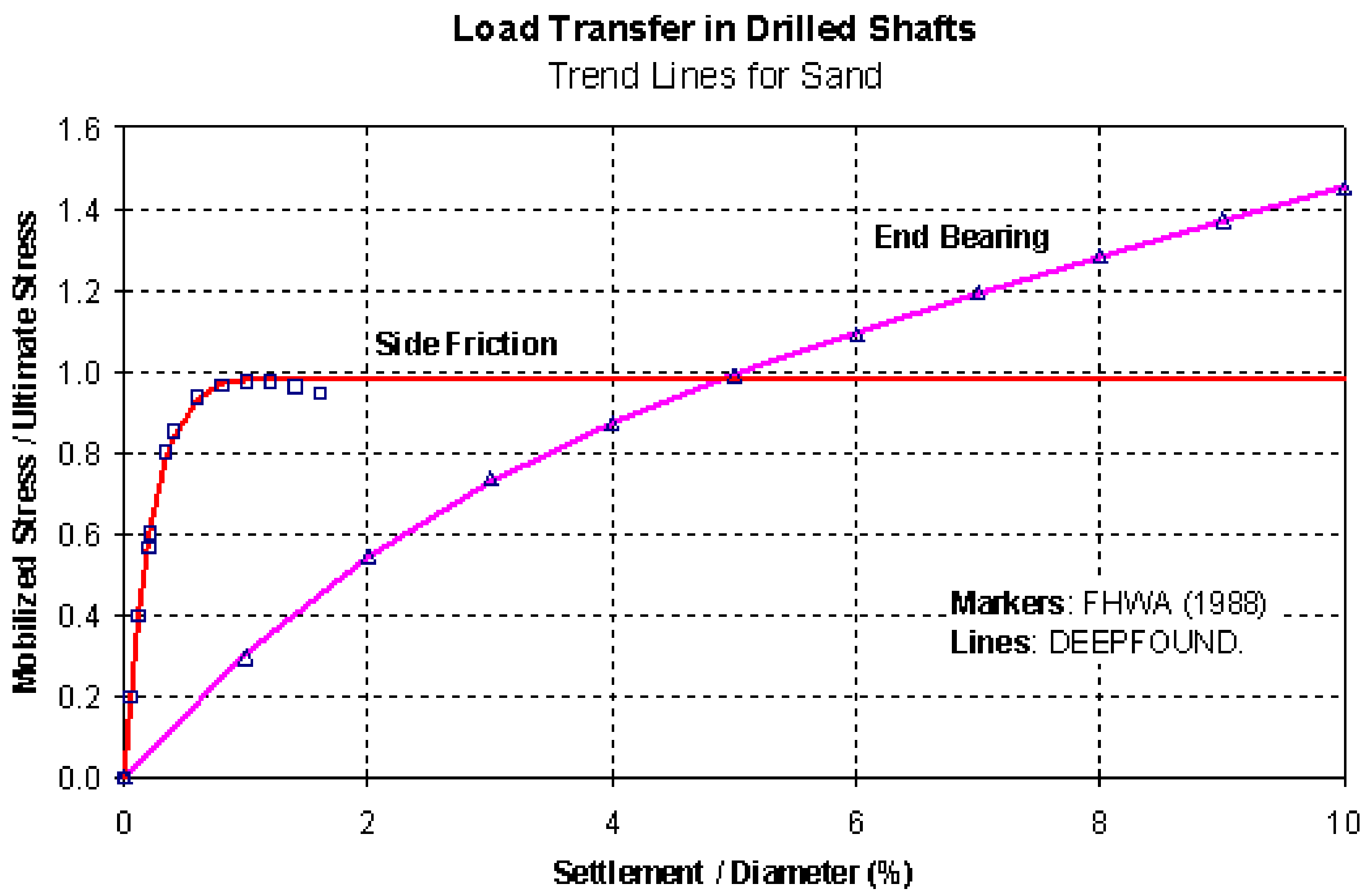
Mobilized End Bearing

q_b/q_{bmax} = -0.0001079* R^4 + 0.0035584* R^3 - 0.045115* R^2 + 0.34861*R

$$R = \frac{S}{B} \times 100 \qquad S = settlement \qquad B = diameter\ of\ shaft$$

Drilled Shafts

Settlement Trend Lines in Clean Sand



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Drilled Shafts

End Bearing in Limestone

$$Q_b = q_{bu} A_b$$

Q_b = ultimate end bearing

q_{bu} = unit end bearing capacity

A_b = shaft base area

(note: q_{bu} is user defined)

Drilled Shaft

Design for Rock Socket

Side shear resistance for limestone

$$f_{su} = 0.5\sqrt{q_u}\sqrt{q_t} \qquad (McVay, 1992)$$

Other methods

$$f_{su} = aq_u^b$$

This equation is a generic form for most of the other correlations. In which *a & b* are empirical parameters based on personal experience in the geographical and geologic areas of the authors.

For example William's: $f_{su} = 1.842 q_u^{0.367}$

Short-Term Settlements in Rock (O'Neill, et. al. 1996)

For side shear resistance - There are six (6) steps to calculate the side resistance in relative to deformations along the side of the rock socket;

1. Find the average E_m and f_{su} along the side of the rock socket

$$E_m(weighted\,average) = \frac{\sum E_{mk} L_k}{\sum L_k} \quad and \quad E_{mk} = 115 q_{uk}$$

$$f_{su} = \frac{\sum f_{su} L_k}{\sum L_k} \quad where \quad f_{su} = ultimate\ side\ friction$$

$$depend\ on\ smooth\ or$$

$$roughness\ of\ socket\ wall$$

$$L_k = k^{th}\ layer\ thickness$$

Short-Term Settlements in Rock (side shear)

2. Calculate Ω

$$\Omega = 1.14\left(\frac{L}{B}\right)^{0.5} - 0.05\left[\left(\frac{L}{B}\right)^{0.5} - 1\right]\log_{10}\left(\frac{E_c}{E_m}\right) - 0.44$$

$$where \qquad E_c(\Psi) = 57\,000\sqrt{q_{uc}}$$

3. Calculate Γ

$$\Gamma = 0.37\left(\frac{L}{B}\right)^{0.5} - 0.15\left[\left(\frac{L}{B}\right)^{0.5} - 1\right]\log_{10}\left(\frac{E_c}{E_m}\right) + 0.13$$

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Short-Term Settlements in Rock (side shear)

4. Find n for "rough" sockets;

$$n = \sigma / q_u$$

where σ = normal stress of concrete

$$= \gamma_c Z_c M$$

$$\gamma_c \approx 130 \text{ pcf or } 20.5 \text{ kN/m}^3$$

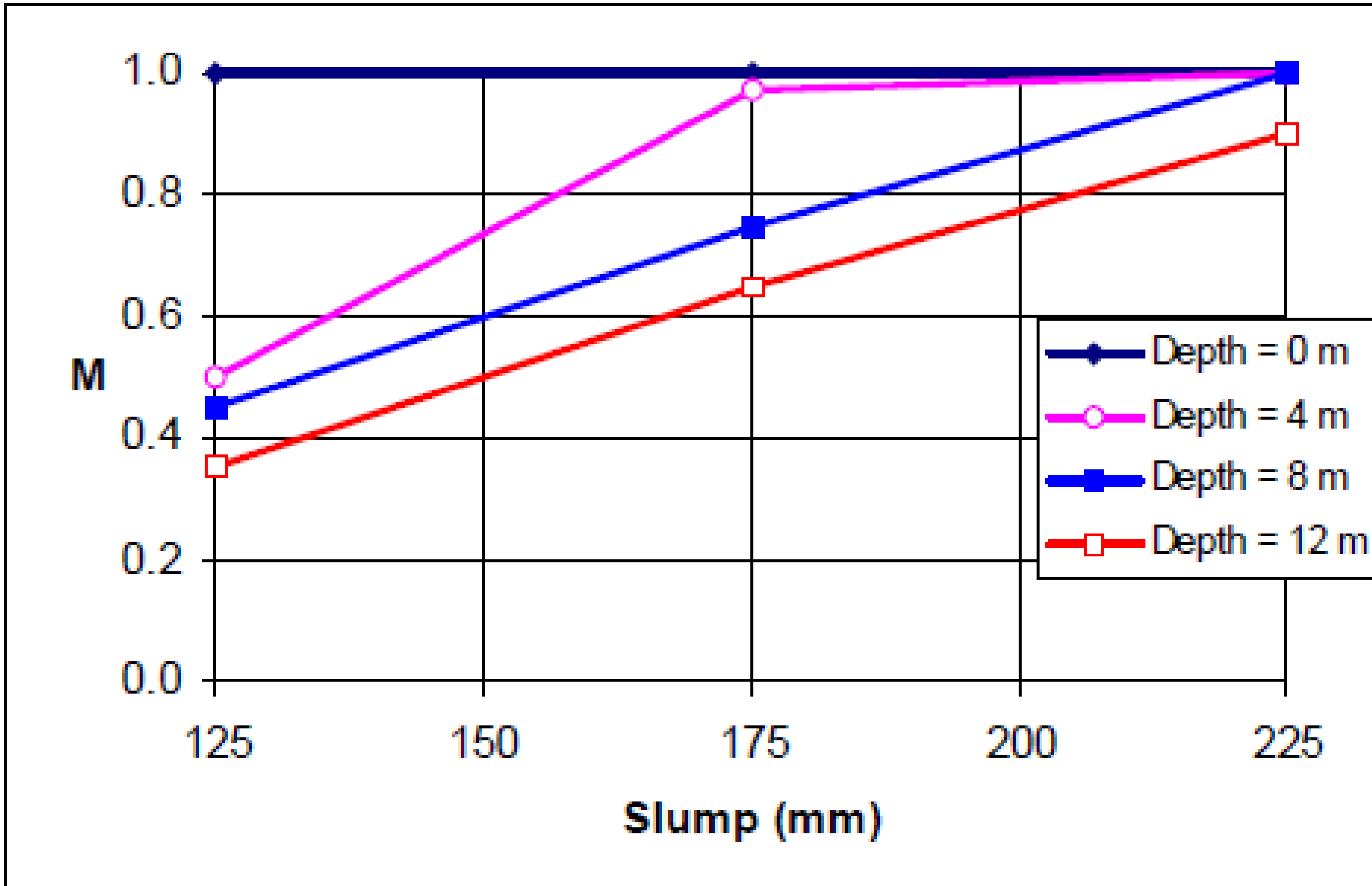
M is a function of concrete slump and socket depth

Short-Term Settlements in Rock (side shear)

Values of M

Socket Depth (m)	Slump (mm)		
	125	175	225
4	0.50	0.95	1.0
8	0.45	0.75	1.0
12	0.35	0.65	0.9

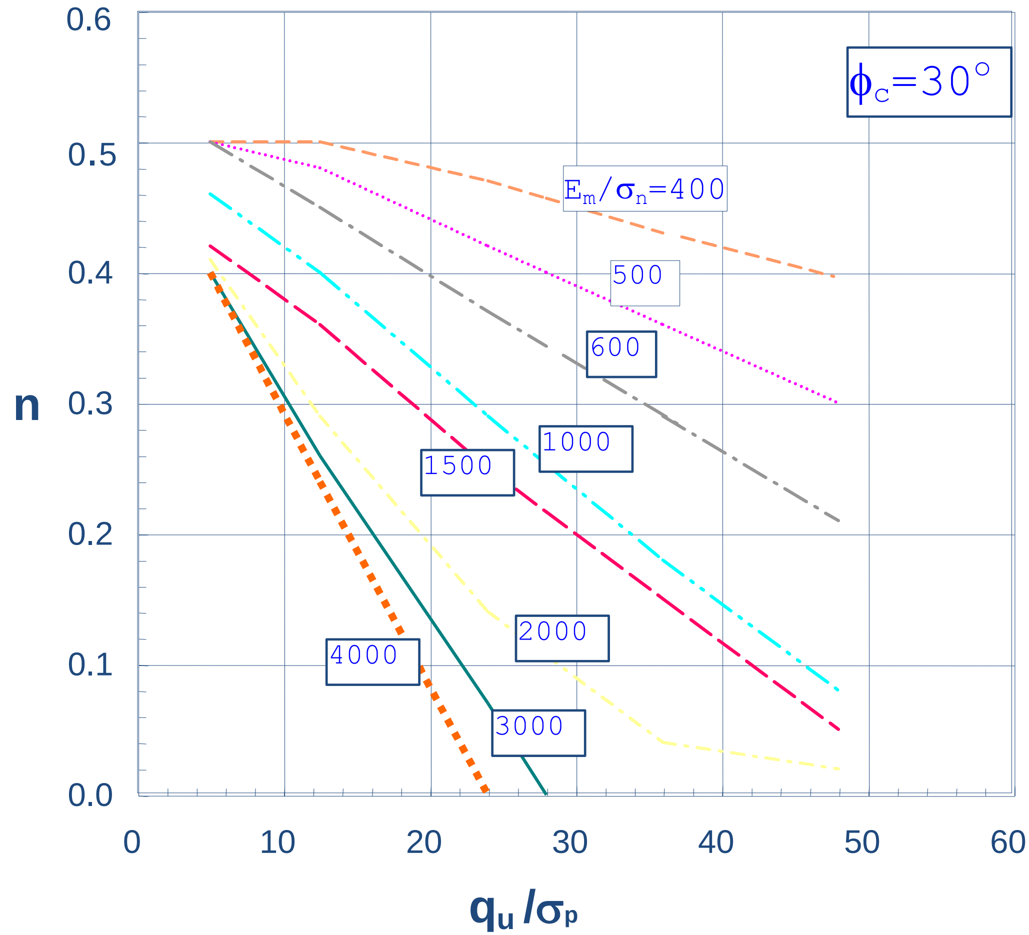
Short-Term Settlements in Rock (side shear)



Short-Term Settlements in Rock (side shear)

For “smooth” socket;

This chart is for $\phi_c = 30^o$ but n is not sensitive to ϕ_c



Short-Term Settlements in Rock (side shear)

5. Calculate Θ_f and K_f

$$\Theta_f = \frac{E_m \Omega}{\pi L \Gamma f_{su}} w_t$$

$$K_f = n + \frac{(\Theta_f - n)(1-n)}{\Theta_f - 2n + 1} \quad < 1$$

$$where \quad w_t = deflection\ at\ top\ of\ the\ rock\ socket$$

6. Calculate the side shear load transfer - deformation

$Q_s = \pi B L \Theta_f f_{su}$ $\quad \Theta_f < n$ $\quad$ (in the elastic zone before slippage)

$Q_s = \pi B L K_f f_{su}$ $\quad \Theta_f > n$ $\quad$ (during interface slippage)

Short-Term Settlements in Rock

End bearing

$$Q_b = \frac{\pi B^2}{4} q_b \qquad q_b = \Lambda w_t^{0.67}$$

$$where \quad \Lambda = 0.0134 E_m \frac{(L/B)}{(1+L/B)} \left[\frac{\left[200(L/B)^{0.5} - \Omega\right]\left[1+(L/B)\right]}{\pi L \Gamma} \right]^{0.67}$$

Layered Soils

Side friction - sum of the side resistance of each soil layer;

End bearing - the resistance of the soil type at the base.

Questions ?

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