Geotechnical Engineering Research Laboratory
University of Massachusetts
Lowell, MA USA

Reliability of Settlement Analysis for Shallow Foundations

14.533 ADVANCED FOUNDATION ENGINEERING
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Samuel G. Paikowsky

GeoDynamica Inc.
Newton, MA USA

Geotechnical Engineering Consultants

AASHTO Bridge Meeting 2007

AASHTO
Subcommittee on
Bridges and Structures

Delaware
It's good being first.
THE FIRST STATE

Dumont Hotel
Wilmington, Delaware
AASHTO and Delaware Department of Transportation

June 8-12, 2007

Samuel G. Paikowsky

UMASS LOWELL

University of Massachusetts
Lowell, MA USA

Geotechnical Engineering Research Laboratory

NCHRP 12-66 OBJECTIVES

Develop procedures for serviceability design of bridge foundations, calibrate them and write AASHTO specifications

Practically, develop new methodology to calibrate serviceability in LRFD and write new specifications.

Research Plan

I. Establish serviceability criteria for bridges under normal operation.

II. Compile large databases for foundation displacements.

III. Determine the analyses methods used for calculating foundation displacements and establish their uncertainty.

IV. Develop methodology and establish LRFD parameters for serviceability.

LSD OVERVIEW

Limit states design (LSD) **was initiated in the 1950's for a more economical design.**

Two types of limit states need to be satisfied :

Ultimate Limit State (ULS)

Factored resistance ≥ Factored load effects

Serviceability Limit State (SLS)

Factored Deformation ≤ Tolerable deformation to remain serviceable

AASHTO Research Perspective

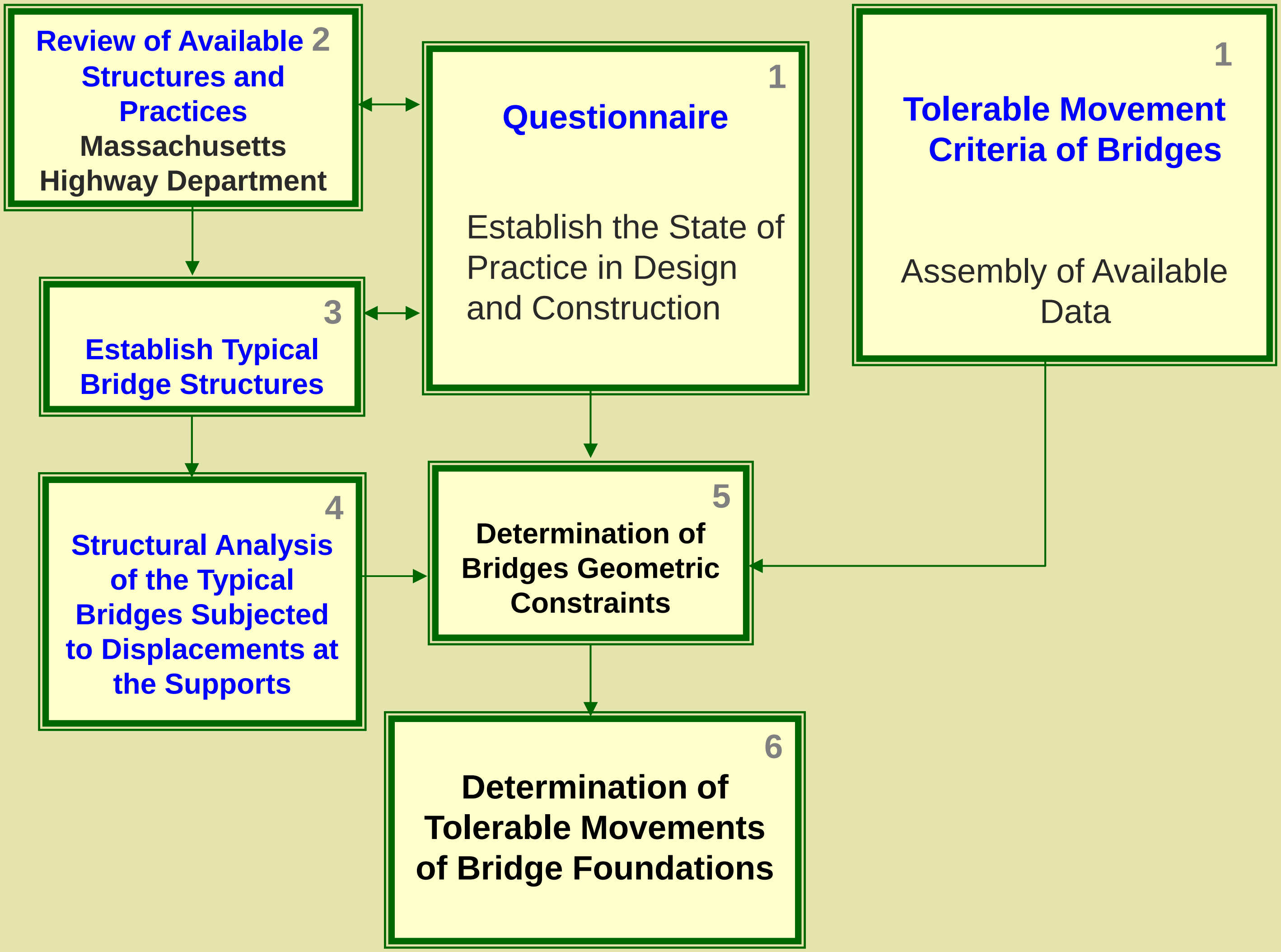
Ultimate Limit State

- **Deep Foundations – NCHRP 12-47 published as NCHRP Report 507 - rework and parts appear in the Specifications**
  (for a pdf file use Google – NCHRP 507)
- **Shallow Foundations – NCHRP 24-31 published as NCHRP Report 651**
  (for a pdf file use Google – NCHRP 651)

Serviceability Limit State

- **Current Presentation – NCHRP 12-66 Final Report submitted fall 2008.**

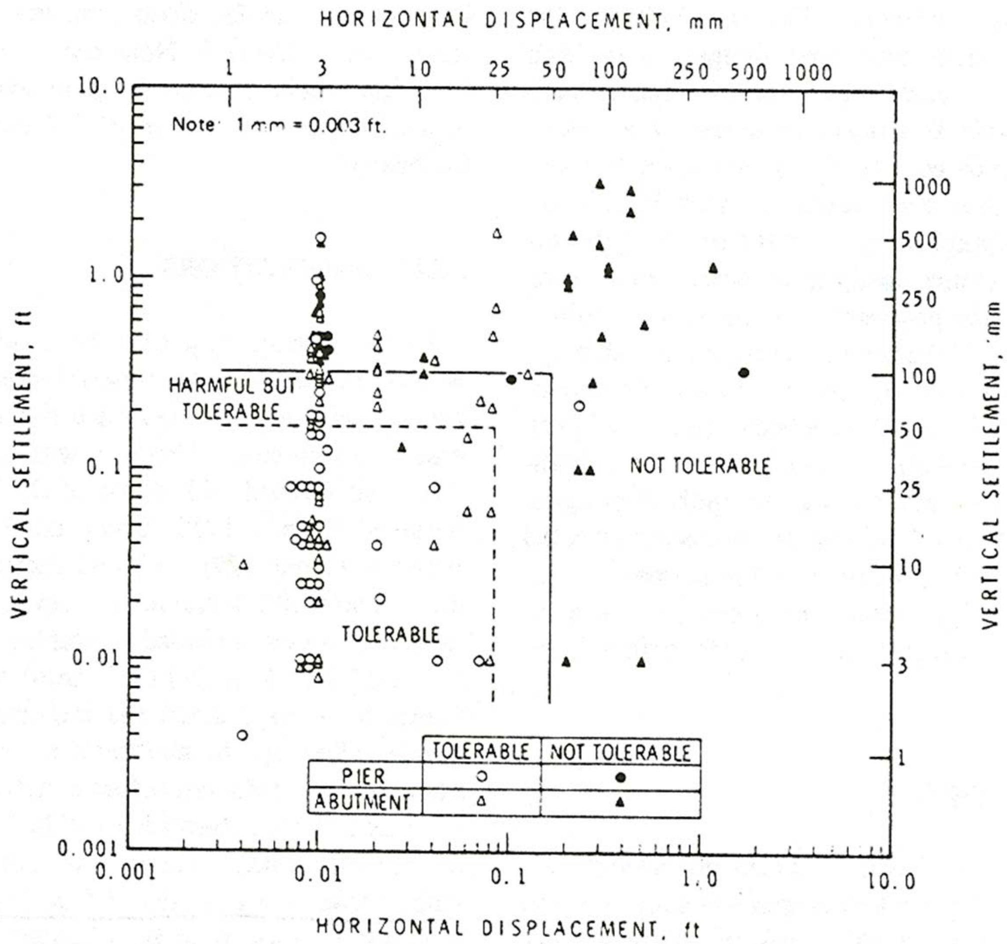
Research Plan for Establishing the Serviceability Criteria for Bridge Foundations Under Normal Operation



Summary of All Data (Bozozuk, 1978)

Vertical Movements	(mm)	(in)
Tolerable	< 50	< 2
Harmful but tolerable	50-100	2-4
Intolerable	> 100	> 4
Horizontal Movements	**(mm)**	**(in)**
Tolerable	< 25	< 1
Harmful but tolerable	25-50	1-2
Intolerable	> 50	> 2

Stermac (1978) Questioned Bozozuk data since they were processed without consideration of bridge type, span length, type of movement (differential vs. total)



Engineering Performance of Bridge Abutments and Piers on Spread Footings (Bozozuk, 1978)

Field Studies Moulton (1986)

Support	Total No.		% Moved
	Observed	Moved	
Abutment	580	439	76
Piers	1068	269	25

Support	Vertical	Horizontal	Combined
Abutments	86%	31%	18%
Piers	87%	19%	6%

Avg. vertical pier movement (2.5in) < abutment avg. (3.7in)
Avg. horizontal pier movement (3.3in) > abutment avg. (2.6in)

Existing AASHTO LRFD Specifications for Serviceability

Section	Foundations	Criteria	Reference
4.4.7.2.5	Footings	$\Delta/\ell < 1/200$ simple span $\Delta/\ell < 1/250$ continuous	Moulton, 1985
10.7.2.2	Pile and Groups	$\delta_h \leq 1.0$inch when δ_h & δ_v combined $\delta_h \leq 1.5$inch only δ_h	

Comments:

1. **No vertical displacements**
2. **No consideration to bridge type, span length, and rigidity**
3. **Criteria more restrictive than that recommended by Moulton**

Superstructure of Bridges

Major Findings of the Questionnaire

- **8,281 new/replacement bridges were built and 5,421 bridges were rehabilitated in the Responding States over a 5 year period (1999-2003)**
- **Summary**
  - **Integral Abutment – 46.6% (single 10.7%, multispan 35.9%)**
  - **Multispan – 36.0% (simple 8.5%, continuous 27.5%)**
  - **Single Span Simple – 14.4%**
  - **All others – 2.5%**

Number of Bridges per Type and Span Selected as "Typical" Bridges

(represent 97.5% of Constructed Bridges)

Span Length	Bridge Type / Case No.		
	Simple Single & Multi-Span	Continuous Multi-Span	Integral Abutment
Short 20 – 125ft	#1	#5	#9
*Medium 125 – 400ft	#2, #3	#6, #7	#10, #11
Long > 400ft	#4	#8	#12

* Steel & concrete construction for each medium span length bridge type

Evaluation of Bridge Response for Pier and Abutment Displacements

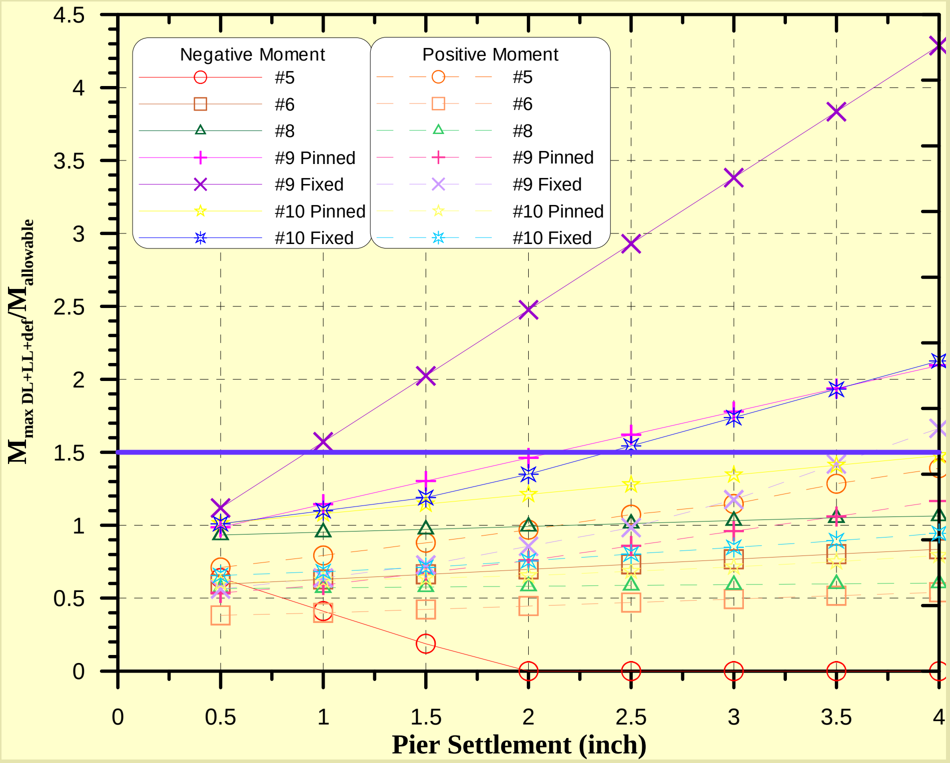
Evaluation of deflections, moments and stresses in all 12 bridges under, dead loads, live loads, support settlement and their combinations

Bridge #9, Fitchburg MA

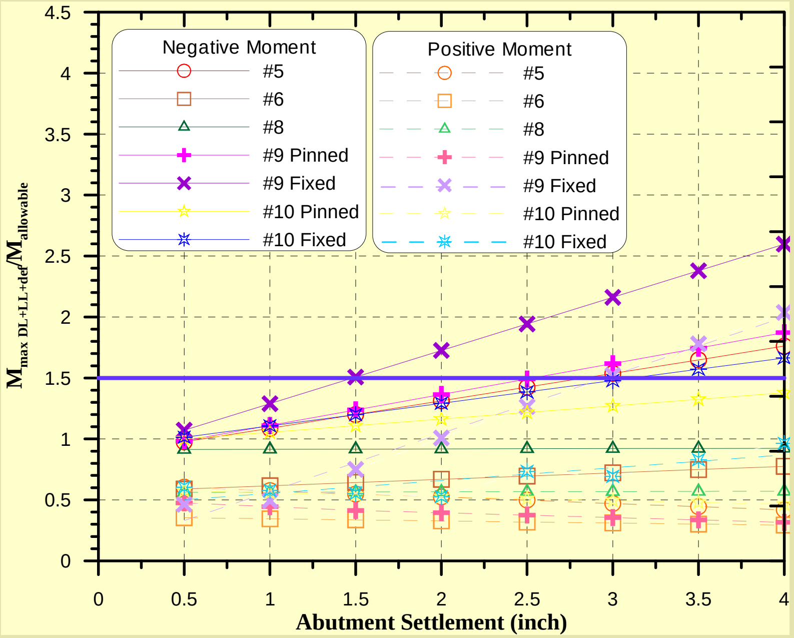
(3-Span Short Steel Girders, Concrete deck Integral Abutment Bridge)

Evaluation of Bridge Response for Pier and Abutment Displacements

Ratios of Moments Induced by Bridge Support Movement, Dead and Live Loads Combined over Allowable moment (Mmax DL+LL+def/Mallowable) as a Result of (a) Pier Settlement and (b) Abutment Settlement of Steel Bridges



(a) Pier



(b) Abutment

Summary of Proposed New Serviceability Criteria

Criteria	Bridge Type	Limit State	Limitations	Comments
Angular distortion	Simple Support	$\Delta/\ell < 1/200$	• $\ell \geq 50$ft	• subjected to limit vertical displacements • exc. rigid frame structures • exc. integral abutment bridges assuming pinned connection at the abutment
Angular distortion	Continuous	$\Delta/\ell < 1/250$	• $\ell \geq 50$ft steel	
Abutment differential vert. displacement for bridge lifetime	Steel	$\Delta_{VA} < 3$in	• $\ell \geq 50$ft steel • $I/\ell \leq 20\text{in}^3$	Moulton, 1986, Table 7; Current study Table 4.14
	Concrete	$\Delta_{VA} < 3$in	• $\ell \geq 100$ft	Moulton, 1986, p.58; Current study Table 4.14
Pier differential vert. displacement for bridge lifetime	Steel	$\Delta_{VP} < 2$in	• $\ell \geq 50$ft	Moulton, 1986, Table 7; Current study Table 4.14
	Concrete	$\Delta_{VP} < 2$in		
Abutment differential vert. displacement following bridge completion	Steel	$\Delta_{VA} < 2$in	• $\ell \geq 50$ft	
	Concrete	$\Delta_{VA} < 2$in		
Pier differential displacement following bridge completion	Steel	$\Delta_{VP} < 1.25$in		
	Concrete	$\Delta_{VP} < 1.50$in		
Horiz. displacements	All Substructures	$\Delta_h < 1.5$in	Controlling criteria	AASHTO; Moulton 1986, $\Delta_h < 2.0$in
Horiz. displacements combined with vert. displacements	All Substructures	$\Delta_h < 1.0$in	Controlling criteria	AASHTO; Moulton 1986, $\Delta_h < 1.5$in

Databases

Performance of DP – Compression, Tension, and Lateral / Pile Type/ Soil Type

Performance of Drilled Foundations – Compression, Tension, and Lateral / Construction Type/ Soil Type

Performance of Pile Groups – Vertical / Lateral / Soil Type

Performance of Shallow Foundations – Prototype / Full Scale / Soil Type

Performance of Full Scale Structures – Piers / Abutments

Establishing Methods of Analysis for Calibration

1. **State of Practice – Questionnaire**
   Substructures of Bridges

2. **Existing methods available at the current specifications and related literature**

Substructure of Bridges – Major Findings

Construction

- **Foundation alternatives:**
  62% (75)* driven piles, 21% (11)* In Place Constructed Deep Foundations (IPCDF) and 17% (14)* shallow foundations. * = 1998 questionnaire
- **Shallow foundations:**
  on rock (55%), frictional soil (23%), IGM (19%), and cohesive soils (3%). About half of the shallow foundations built on clay are constructed with ground improvement measures, i.e. only about 0.25% of the total bridge foundations are built on clay with some states indicating they construct shallow foundations on rock only (AK, TN), don't use shallow foundations at all (LA, TX) but utilize the analyses for retaining walls, etc. (TX).

Determination of the Uncertainty in the Displacement Analyses of Foundations

- **Settlement of Shallow Foundations**
- **Lateral Deflection of Piles - single and Groups (pinned and fixed)**
- **Settlement of Piles - single and Groups**

SETTLEMENT ANALYSIS METHODS FOR FOOTINGS ON COHESSIONLESS SOIL

Elastic Method

1. **AASHTO – LRFD Bridge Design Specifications (1998)**

Settlement of footings on cohessionless soils may be estimated using empirical procedures or elastic theory. The elastic settlement of footings on cohessionless soil may be estimated using the following:

$$S_e = \frac{[q_0(1-\nu^2)\sqrt{A}]}{E_s \beta_z} \qquad (1)$$

where q_0 = load intensity (TSF)

A = area of footing (SF)

E_s = Young's modulus of soil taken as specified in Table 1 in lieu of the result of the laboratory tests (TSF)

β_z = shape factor taken as specified in Table 2 (DIM)

ν = Poisson's Ratio taken as specified in Table 1 in lieu of the result of laboratory tests (DIM)

Elastic Method

1. **AASHTO – LRFD Bridge Design Specifications (1998) (cont'd.)**

Unless E_s varies significantly with depth, E_s should be determined at depth of about 1/2 or 2/3 B below the footing. If the soil modulus varies significantly with depth, a weighted average value of E_s (eq.2) maybe used. The following nomenclature shall be used with Table 1:

$$E_s = \frac{\sum E_{s(i)} \Delta z}{\bar{z}} \qquad (2)$$

Elastic Method

1. **AASHTO – LRFD Bridge Design Specifications (1998) (cont'd.)**

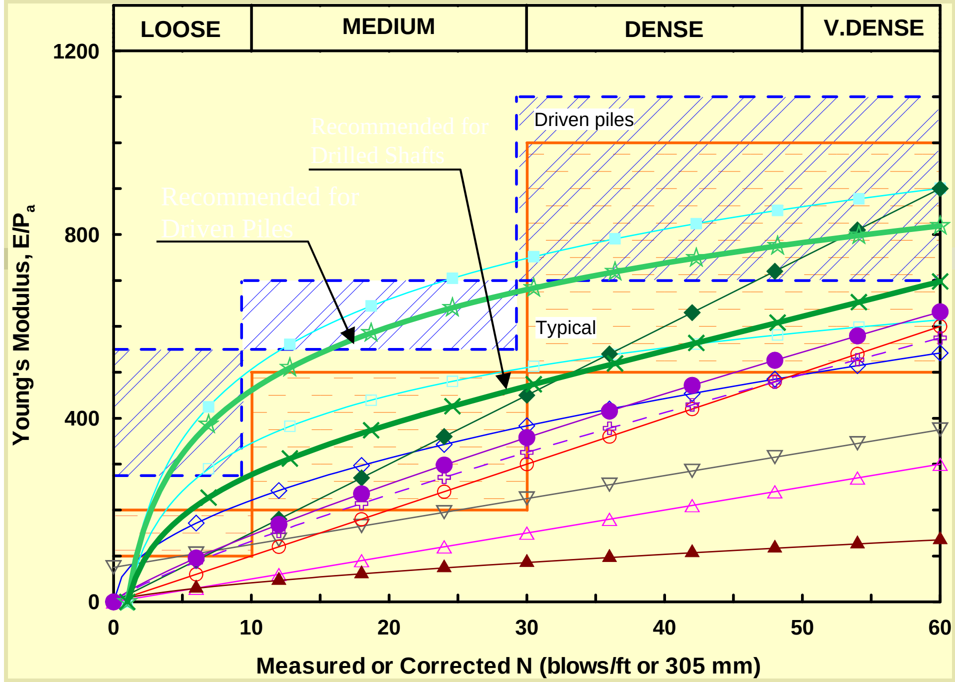
Table 1. Elastic Constants of Various Soils Modified after U.S. Department of the Navy (1982) and Bowels (1988) (AASHTO Table 10.6.2.2.3b-1)

Soil Type	Typical Range of Values: Young's Modulus (tsf)	Poisson's Ratio, ν (dim)	Estimating E_s from N: Soil Type	E_s (tsf)
clay:				
soft sensitive	25-150	0.4-0.5	Silts, sandy silts, slightly cohesive mixtures	$4N_1$
Medium stiff to stiff	150-500	(undrained)	Clean fine to medium sands & slightly silty sands	$7N_1$
Very stiff	500-1000		Coarse sands and sand with little gravel	$10N_1$
			Sandy gravel and gravels	$12N_1$
Loss Silt	150-600	0.1-0.3	Sandy gravel and gravels	$12N_1$
	20-200	0.3-0.35		
Fine Sand:			**Estimating E_s from S_u**	
Loose	80-120		Soft sensitive clay	$400S_u$-$1{,}000S_u$
Medium dense	120-200	0.25	Medium stiff to stiff clay	$1{,}500S_u$-$2{,}400S_u$
Dense	200-300		Very stiff clay	$3{,}000S_u$-$4{,}000S_u$
Sand:				
Loose	100-300	0.20-0.35		
Medium dense	300-500			
Dense	500-800	0.30-0.40		
Gravel:			**Estimating Es from q_c**	
Loose	300-800	0.2-0.35		
Medium dense	800-1,000		Sandy Soil	$4q_c$
Dense	1,000-2,000	0.3-0.4		

Notes: N = Standard Penetration Test (SPT) resistance; N_1 = SPT corrected for depth

S_u = undrained shear strength (TSF); q_c = cone penetration resistance (TSF)

Young's Modulus of Sands (E_s) vs. Blow Count



Legend Key	Relations	Soil Type	Reference	Comment
▽	$E_s / p_a = 0.5(N+15)$	NC Sand	General Sources, see Bowles (1996)	$N = N_{55}$
◇	$E_s / p_a = 70\sqrt{N}$	NC Sand	Denver (1982)	$N = N_{55}$
□	$E_s / p_a = 150\ln(N)$	NC Sand	USSR (See Bowles, 1996)	N should be estimated as N_{55}, and N may not be standard blow count
■	$E_s / p_a = 220\ln(N)$	NC Sand	USSR (See Bowles, 1996)	$N = N_{55}$. N may not be the standard blow count
△	$E / p_a = 5N_{60}$	Sands with fines	Kulhawy & Mayne (1990)	
○	$E / p_a = 10N_{60}$	Clean NC Sand	Kulhawy & Mayne (1990)	
◆	$E / p_a = 15N_{60}$	Clean OC Sand	Kulhawy & Mayne (1990)	
●	$E_D / p_a = 22N^{0.82}$	Piedmont Sandy Silts	Using Mayne & Frost (1989)	Recommended by O'Neill & Reese (1999) for use with drilled shaft elastic analysis in cohesionless IGM. E_D is the modulus measured in the dilatometer test (DMT).
✚ (dashed)	$E / p_a = 20.02N^{0.82}$	Piedmont Sandy Silts	Mayne & Frost (1989)	E_D is replaced by E_s through the relation: $E_D=E_s/(1-v^2)$, & v=0.3.
▲	$E_{PMT} / p_a = 9.08N^{0.66}$	Sand	Ohya, et al (1982)	E_{PMT} is the modulus measured in the pressuremeter test (PMT), and is often presumed to be roughly equivalent to Young's modulus E.
☆ (thick)	$E_s / p_a = 200\ln(N)$	Sand	Current study Driven Piles	For N>60 use N=60. Recommended for driven piles
✕ (thick)	$E_s / p_a = 112e^{0.007N}\ln(N)$	Sand	Current study Drilled Shafts	For N>60 use N=60. Curve best fit of all information for drilled shaft.

For driven piles

$E_s / p_a = 200\ln(N),\ \ N \le 60$

For drilled shafts

$E_s / p_a = 112e^{0.07}\ln(N),\ N \le 60$

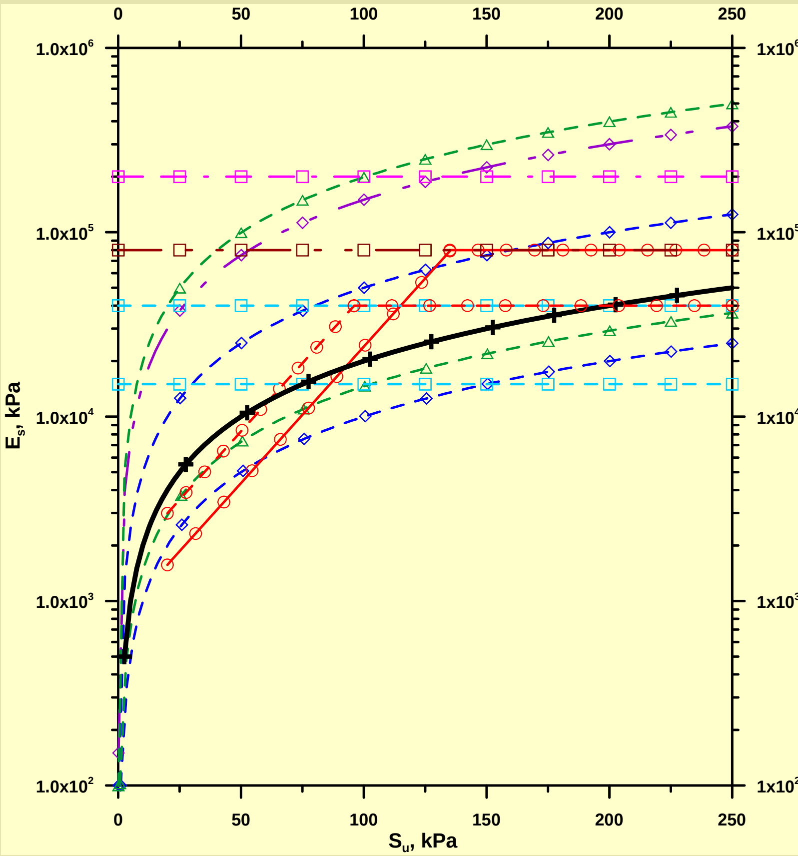
where:

p_a = atmospheric pressure = 0.1 MPa

E_s = Young's modulus of soils

N = corrected blow count in SPT tests for 60% energy and vertical effective stresses

Young's Modulus of Clay (E_s) vs. Undrained Shear Strength



Legend Key	Relations	Soil Type	Reference	Comment
	$E_s = (100 \sim 500)s_u$	I_p > 30 or organic	General resource, see Bowles (1996)	Lines represent upper and lower range.
	$E_s = (500 \sim 1500)s_u$	I_p < 30 or stiff	General resource, see Bowles (1996)	Lines represent upper and lower range.
	$E_s = Ks_u$ $K = 4200 - 142.54I_p + 1.73I_p^2 - 0.0071I_p^3$	General clay	General resource, see Bowles (1996)	Use 20%≤I_p≤100% and round K to nearest multiple of 10. Lines represent upper and lower range.
		Clay	Poulos & Davis (1990)	For driven piles. Drained condition.
		Clay	Poulos & Davis (1990)	For bored piles. Drained condition.
	$E_u = 15000 \sim 40000$	Soft clay	Kulhawy & Mayne (1990)	Lines represent upper and lower range. Undrained condition.
	$E_u = 40000 \sim 80000$	Medium clay	Kulhawy & Mayne (1990)	Lines represent upper and lower range. Undrained condition.
	$E_u = 80000 \sim 200000$	Stiff clay	Kulhawy & Mayne (1990)	Lines represent upper and lower range. Undrained condition.
	$E_s = 200S_u$	Clay	Current study	Reasonable approximation for all piles in clay

Young's Modulus

In Sand

For driven piles

$$E_s / p_a = 200\ln(N), \quad N \le 60 \qquad (6.2)$$

For drilled shafts

$$E_s / p_a = 112e^{0.07}\ln(N), \quad N \le 60 \qquad (6.3)$$

where:

p_a = atmospheric pressure = 0.1 MPa

E_s = Young's modulus of soils

N = corrected blow count in SPT tests for 60% energy and vertical effective stresses

Both equations are limited by the value of Es for N=60, i.e. for N>60 use N=60. The equation 6.3 has the combination of exponential and logarithmic formats to overcome the overestimation of E when N<10 and the underestimation of E when N>30.

In Clay

$$E_s = 200S_u \qquad (6.4)$$

where:

E_s = Young's modulus of soils

S_u = undrained shear strength

Elastic Method

1. **AASHTO – LRFD Bridge Design Specifications (1998) (cont'd.)**

Table 2. Elastic Shape and Rigidity Factor, Kulhawy (1983) (AASHTO table 10.6.2.2.3b-2)

L/B	Flexible, b_z (Average)	b_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.1
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

Elastic Method

1. **AASHTO – LRFD Bridge Design Specifications (1998) (cont'd.)**

For loads eccentric to the centroid of the footing, a reduced effective area, B' x L', within the confines of the physical footing shall be used in geotechnical design for settlement or bearing resistance. The design bearing pressure on the effective area shall be assumed to be uniform. The reduced effective area shall be concentric with the load. The reduced dimensions for an eccentrically loaded rectangular footing may be taken as:

$$B' = B - 2e_B \qquad (3)$$

$$L' = L - 2e_L \qquad (4)$$

where e_B = eccentrically parallel to dimension B (FT)

e_L = eccentrically parallel to dimension L (FT)

Elastic Method

1. **AASHTO – LRFD Bridge Design Specifications (1998) (cont'd.)**

Footing under eccentric loads shall be designed to ensure that:

- The factored bearing resistance is not less then the effects of factor loads, and
- For footings on soils, the eccentricity of the footing evaluated based on factored loads, is less than ¼ of the corresponding footing dimension, B or L.

Elastic Method

1. **AASHTO – LRFD Bridge Design Specifications (1998) (cont'd.)**

For structural design of an eccentrically loaded foundation, a triangle or trapezoidal contract pressure distribution based on factored loads shall be used. The reduced dimensions for a rectangular footing are shown in Figure 1.

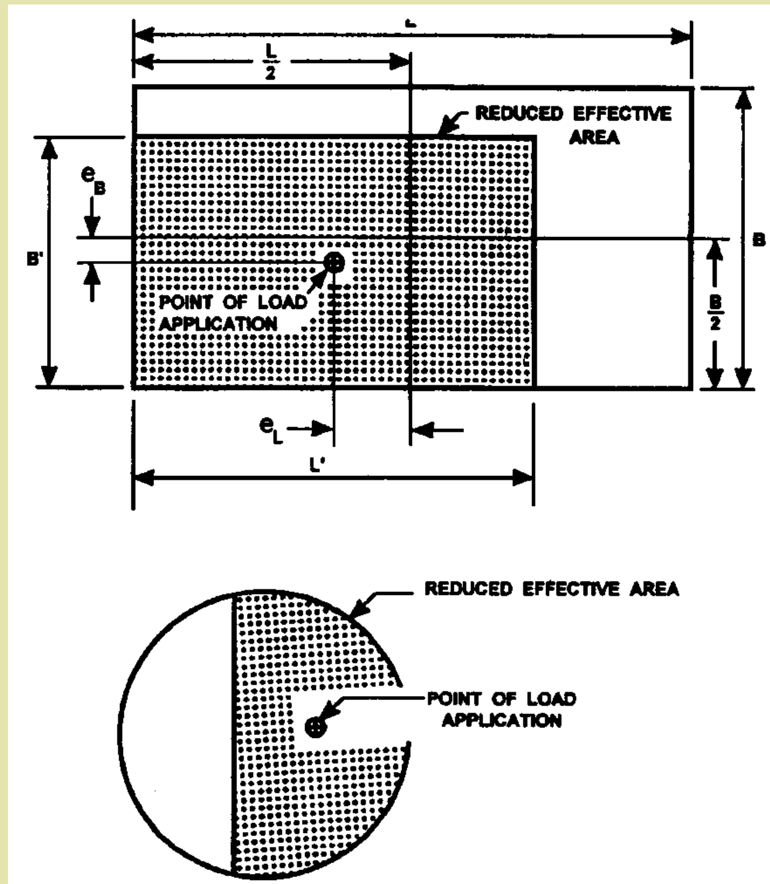


Figure 1. Reduced Footing Dimensions

Elastic Method

2. Schmertmann (1970)

Schmertmann (1970) proposed a method for calculating to settlements of shallow foundations on sands by subdividing the compressible zone beneath the footing into individual layers and then summing the settlement of each sublayer. The method relies heavily on an assumed vertical strain distribution which develops beneath the footing. As presented originally by Schmertmann (1970), this method is often refereed to as “2B-0.6” method which described the approximate strain influence diagram proposed by Schmertmann to calculate settlement over a zone of influence equal to 2B below the footing (Figure 2).

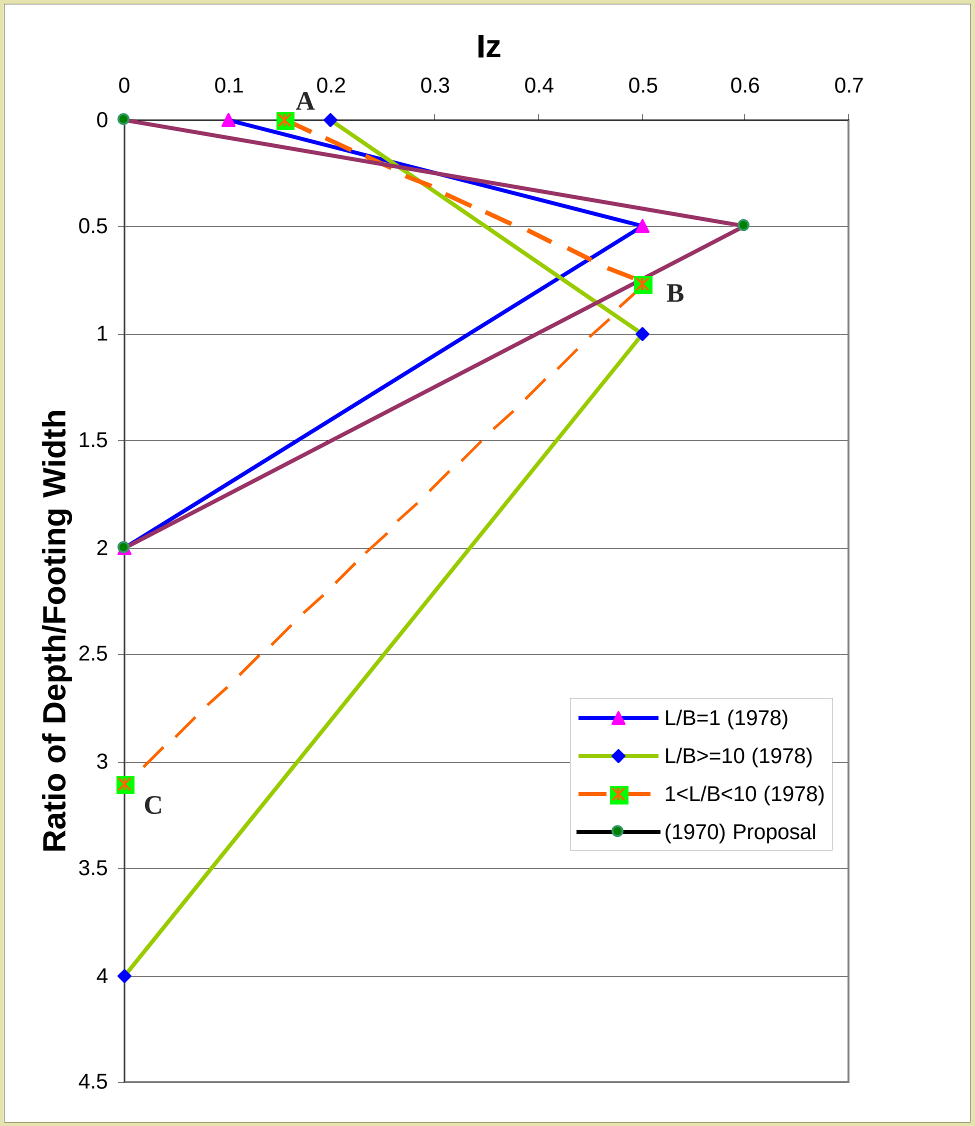


Figure 2. Variation of Iz by different depth Z Chart (Schmertmann, 1970, 1978 and Das, 2004,)

Elastic Method

3. Schmertmann and Hartman (1978)

The settlement of granular sand can also be evaluated by the use of a semi-empirical strain influence factor proposed by Schmertmann and Hartman (1978).

$$S_e = C_1 C_2 (\bar{q} - q) \sum_{i=1}^{n} \frac{I_{zi}}{E_{si}} \Delta z_i \qquad (5)$$

where I_{zi} = strain influence factor for layer i

C_1 = a correction factor for the depth of foundation embedment = $1 - 0.5[q/(\bar{q} - q)]$

C_2 = a correction factor to account for creep in soil = 1 + 0.2 log(time in years/0.1)

$\bar{q}$ = stress at the level of the foundation

q = initial effective overburden pressure at the foundation level, rD_f

E_{si} = soil modulus for layer i, recommended using a weighted average of E_s (eq. 2)

Δz_i = thickness of layer of constant E_{si}

i = layer i

n = total number of layers

Elastic Method

3. Schmertmann and Hartman (1978) (cont'd.)

The variation of the strain influence factor width below the foundation is shown in Figure 3a, while Table 3 shows the strain influence factor at different depth, (B is the width of the foundation, L is the length of the foundation). Figure 2 shows the calculation of elastic settlement by using the strain influence factor chart.

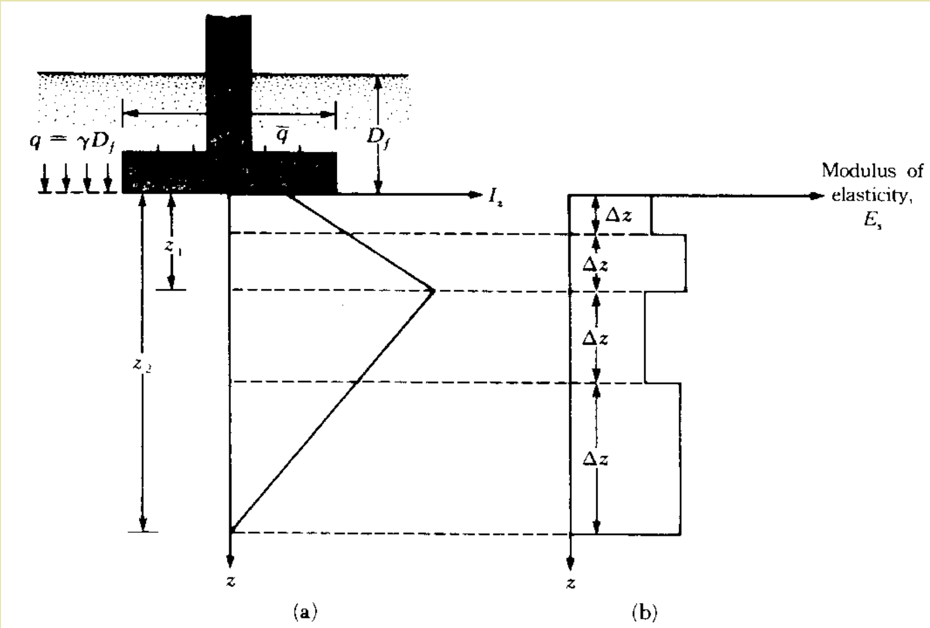


Figure 3. Calculation of elastic settlement by using the strain influence factor (Das, 2004)

Elastic Method

3. Schmertmann and Hartman (1978) (cont'd.)

Table 3. Variation of I_z by different depth z

	z	I_z (Das, 2004)	I_z (Schmertmann, 1978)
square or circular foundations	0.0	0.1	0.1
	$z=z_1=0.5B$	0.5	$0.5+0.1[\Delta q/\sigma'_{vp}]^{0.5}$
	$z=z_2=2B$	0.0	0.0
Foundations with L/B^{3}10	0.0	0.2	0.2
	$z=z_1=B$	0.5	$0.5+0.1[\Delta q/\sigma'_{vp}]^{0.5}$
	$z=z_2=4B$	0.0	0.0

Note: Δq = net applied footing stress

σ'_{vp} = initial vertical effective stress at maximum I_z for each loading case (i.e., 0.5B for axisymmetric and B for plane strain)

The use of eq.5 requires the evaluation of the modulus of elasticity with depth (Figure 3). This evaluation can be made by using the standard penetration test numbers or the cone penetration resistances. The soil is divided into several layers to a depth of $z = z_2$, and the elastic deformation of each layer is estimated. The sum of the deformation of all layers equals the immediate settlement S_e

In Situ Standard Penetration Test (SPT)

1. D'Appolonia et al. (1970)

In the closure to their 1968 ASCE article, D'Appolonia et al. (1970) suggested an alternative method for predicting settlement which is based more or less on an elastic solution. The method requires an estimate of the modulus of compressibility of the soil, M, which is obtained from SPT blow count. The settlement is calculated from the general elastic solution equation:

$$s = {(qBI)}/{M} \qquad (6)$$

where: s = settlement (in ft.)
q = footing stress (in tsf)
B = footing width (in ft)
I = influence factor
M = modulus of compressibility (in tsf)

In Situ Standard Penetration Test (SPT)

1. **D'Appolonia et al. (1970) (cont'd.)**

The influence factor I in eq.6 is the product of two factors, $(\mu_0)(\mu_1)$, which account for the geometry and the depth of the footing and the depth to an incompressible layer. The factors μ_0 and μ_1 were developed by Janbu et al. (1956), modified by Christian & Carrier (1978), see Figure 4.

$I = (\mu_0)(\mu_1)$

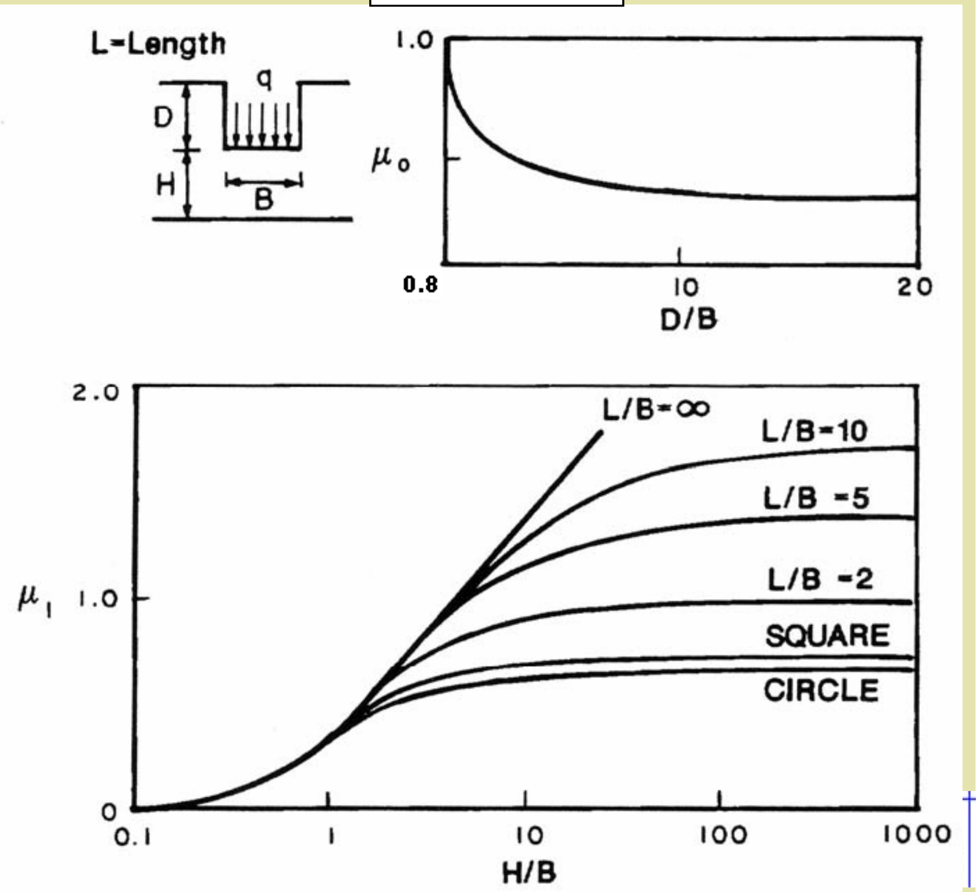


Figure 4. Correction Factors for Embedment and Layer Thickness (Christian & Carrier, 1978)

In Situ Standard Penetration Test (SPT)

1. D'Appolonia et al. (1970) (cont'd.)

The blow count value is taken as the average uncorrected value obtained between the base of the footing and a depth of B below the footing. No other correction factor is applied. The soil modulus of compressibility is obtained from the SPT blow count as:

$$M = 196 + 7.9(N) \text{ (in tsf) for NC sand} \quad (7)$$

$$M = 416 + 10.9(N) \text{ for OC sand} \quad (8)$$

In Situ Standard Penetration Test (SPT)

1. D’Appolonia et al. (1970) (cont’d.)

Figure 5 present the original correlations proposed by D’Appolonia et al. (1970).

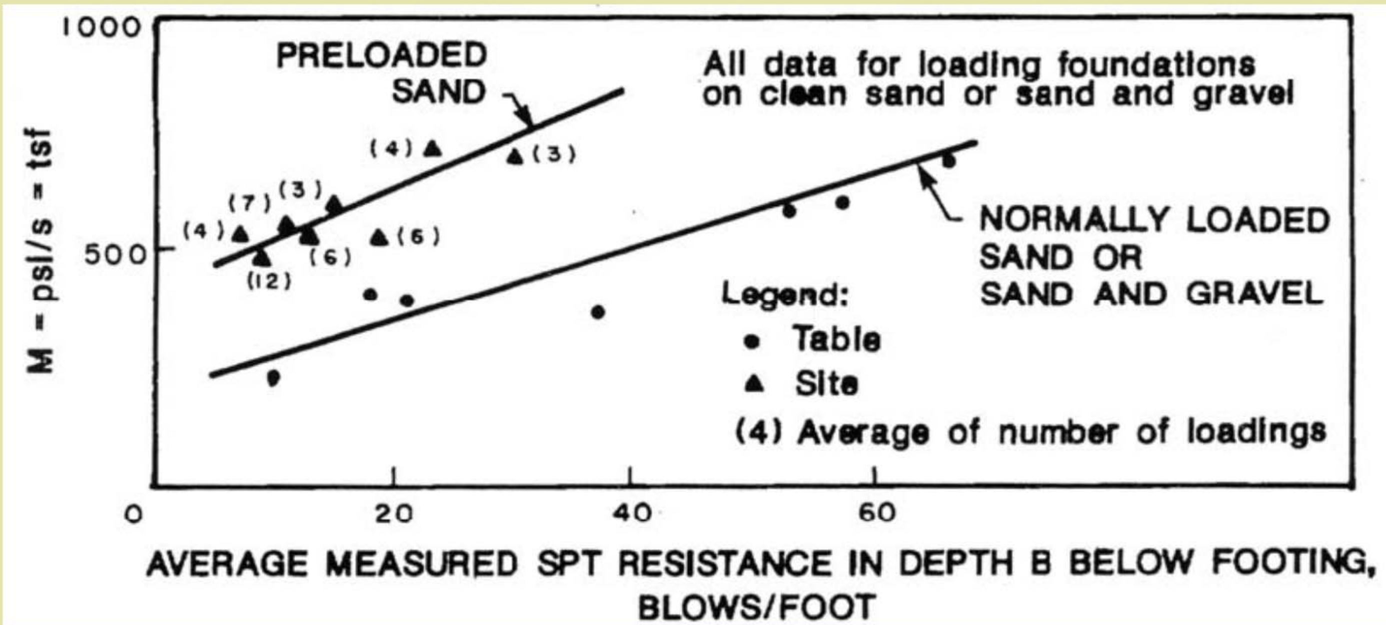


Figure 5. Modulus of Compressibility (D’Appolonia, 1968, 1970)

Note: In the above Figure, “Table” refers to a tabulation of load versus settlement data from seven cases histories, including six bridge footings, by D’Appolonia et al., (1970), and “Site” refers to the load versus settlement data obtained by D’Appolonia at al., (1968) at a large steel mill site in north Indiana.

In Situ Standard Penetration Test (SPT)

2. Hough (1959, 1967)

Basic Equation:

$$\rho = \sum_{0}^{z} \left(\frac{1}{C}\right) \Delta z \log\left(\frac{\bar{\sigma}_{v0} + \Delta\bar{\sigma}_{v}}{\bar{\sigma}_{v0}}\right) \qquad (9)$$

where C = bearing capacity index = $\frac{1+e_0}{C_c}$

e_0 = initial void ratio

C_c = virgin compression index

Δz = layer thickness

$\bar{\sigma}_{v0}$ = initial effective overburden pressure at mid-height of layer

$\Delta\bar{\sigma}_{v}$ = change in effective vertical stress at layer mid-height

In Situ Standard Penetration Test (SPT)

2. Hough (1959, 1967) (cont'd.)

The total settlement by the Hough method is calculated as follows:

a) Corrected SPT blowcounts for overburden stress using Figure 6.

b) Determine bearing capacity index (C') from Figure 7 using corrected SPT blowcounts, N', determined in Step a.

c) Subdivide subsurface soil profile into approximately 3-m (10-ft) layers based on stratigraphy to a depth of about three times the footing width.

d) Calculate the effective vertical stress, σ'_{v0}, at the midpoint of each layer and the average bearing capacity index for that layer.

e) Calculate the increase in stress at the midpoint of each layer, $\Delta\sigma'_{v}$, using 2:1 method (Figure 8).

In Situ Standard Penetration Test (SPT)

2. Hough (1959, 1967) (cont'd.)

The total settlement by the Hough method is calculated as follows:

f) Calculate the settlement in each layer, Δz, under the applied load using the following formula:

$$\Delta H = \frac{1}{C'} \Delta z \log\left(\frac{\bar{\sigma}_{v0} + \Delta\bar{\sigma}_v}{\bar{\sigma}_{v0}}\right) \qquad (10)$$

g) Sum the incremental settlement to determine the total settlement.

In Situ Standard Penetration Test (SPT)

2. Hough (1959, 1967) (cont'd.)

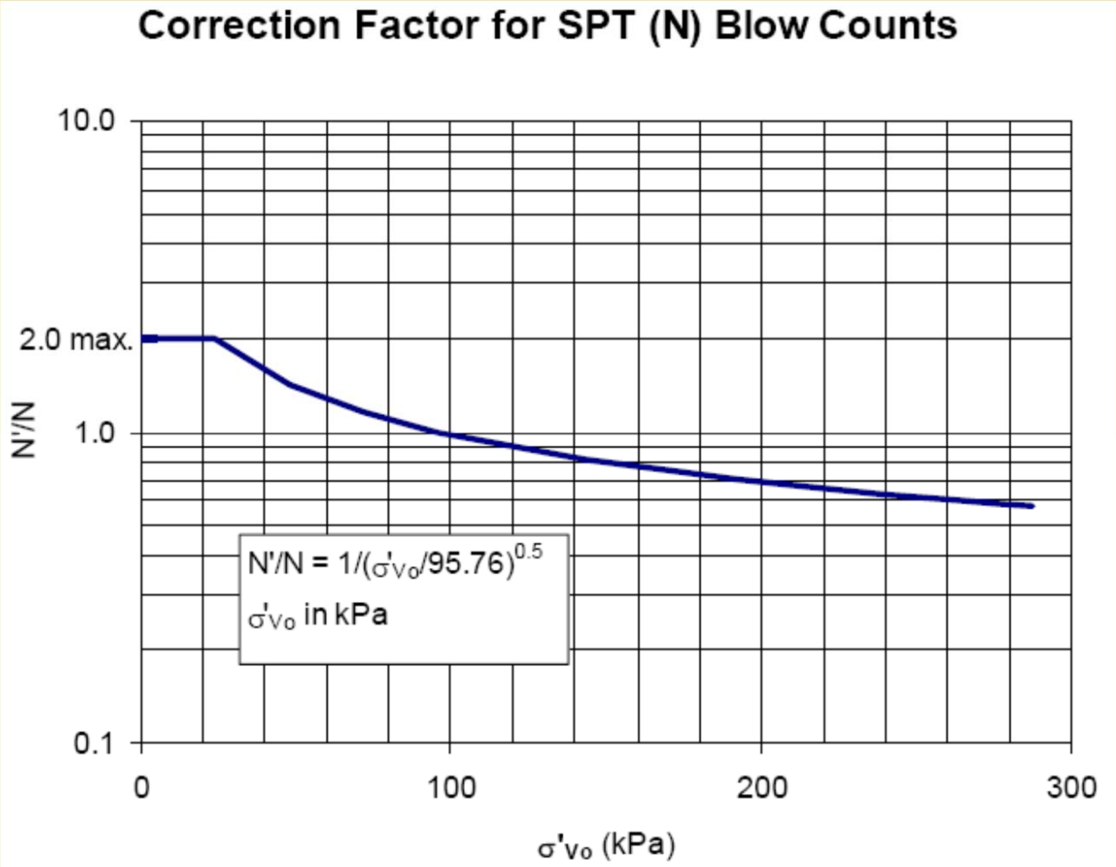


Figure 6. Corrected SPT (N) versus Overburden Pressure (after Liao & Whitman , 1986)

In Situ Standard Penetration Test (SPT)

2. **Hough (1959, 1967) (cont'd.)**

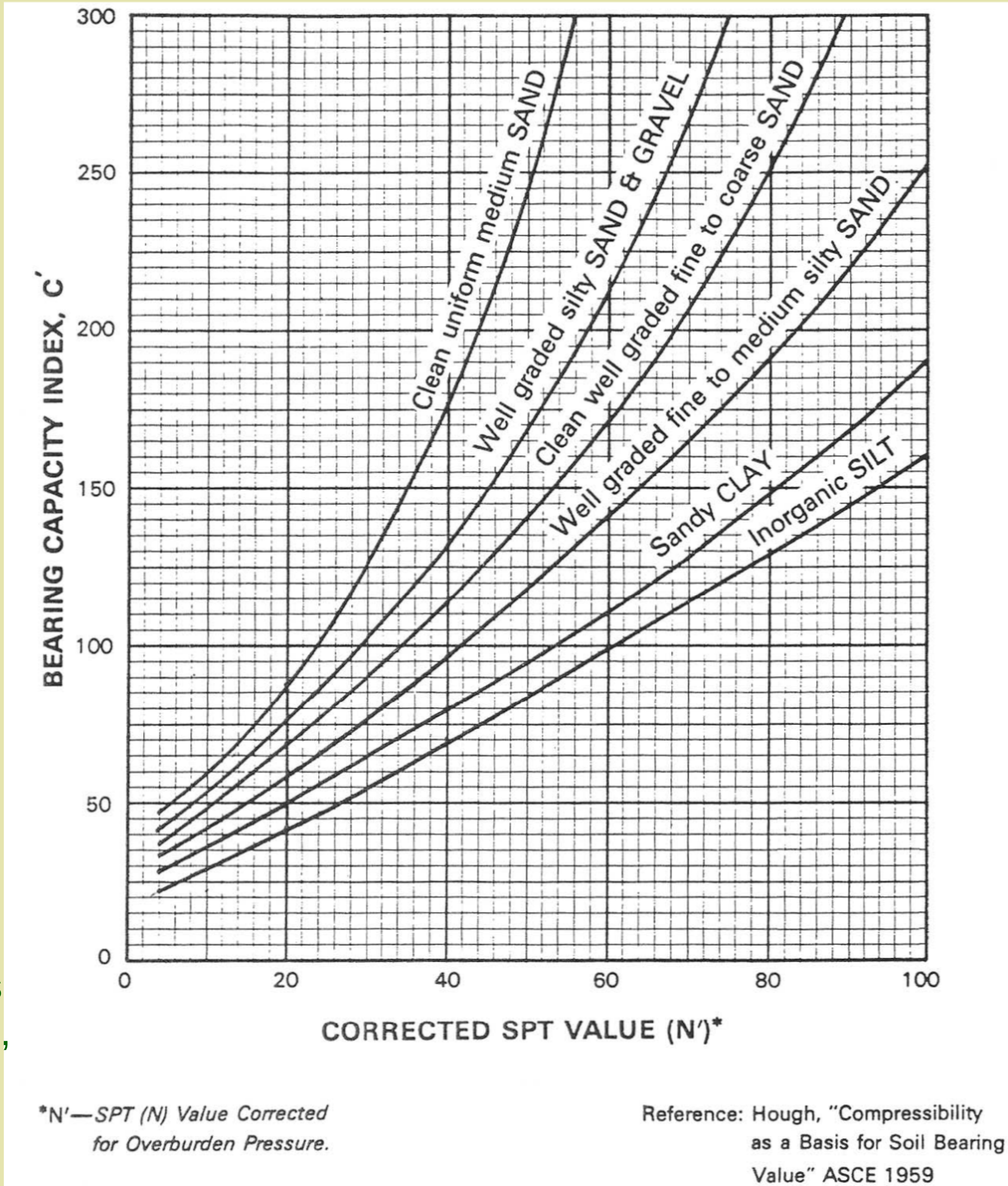


Figure 7. Bearing Capacity Index versus Corrected SPT (Cheney & Chassie, 2000, modified from Hough, 1959)

In Situ Standard Penetration Test (SPT)

2. Hough (1959, 1967) (cont'd.)

Calculate the increase in stress at the midpoint of each layer using the 2:1 method (Chen and McCarron, 1991) as shown in Figure 8. This distribution can be computed as a function of applied stress according to:

$$\frac{\Delta\bar{\sigma}_v}{q} = \frac{B \times L}{(B+z)(L+z)} \qquad (11)$$

where: $\Delta\bar{\sigma}_v$ = change in vertical stress at depth Z below the footing bearing elevation

q = stress applied by the footing at the bearing elevation

z = depth below footing bearing elevation to point of interest, usually the midpoint of a soil layer or sublayer where a settlement computation is to be made

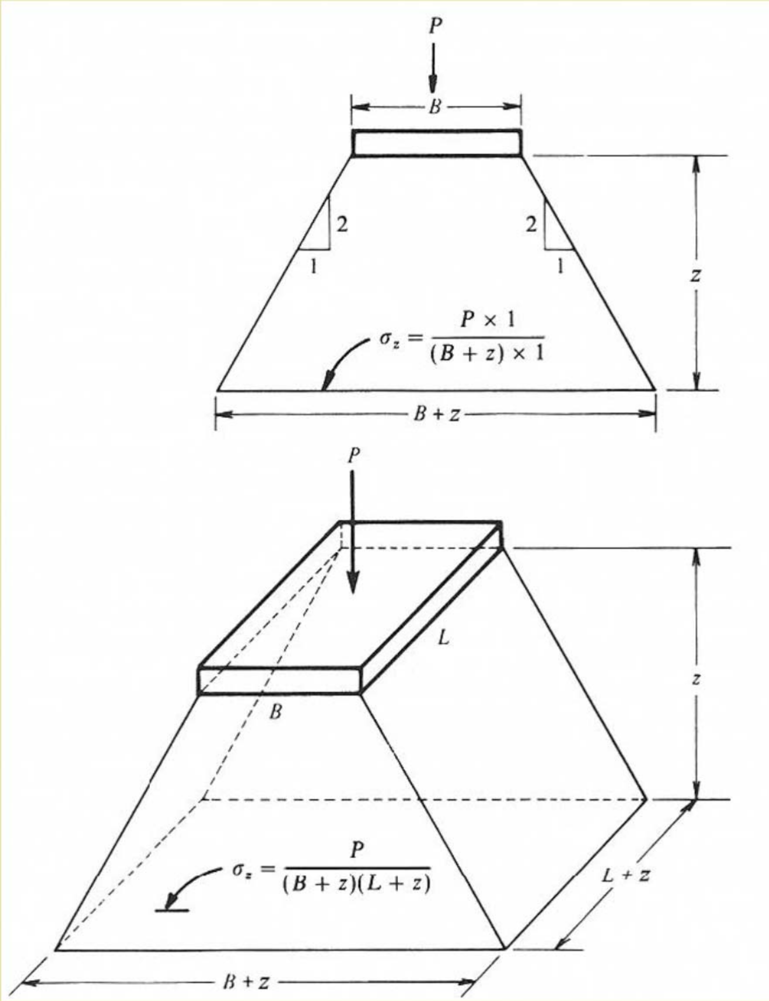
B = width of footing

L = length of footing

In Situ Standard Penetration Test (SPT)

2. Hough (1959, 1967) (cont'd.)

Figure 8. Distribution of Vertical Stress by 2:1 Method (Chen and McCarron, 1991)



In Situ Cone Penetration Test (CPT)

1. Schmertmann (1970)

Using eq.5 which is previously prepared in section 1.2, the soil modulus is estimated using CPT results in the following way. Relationship between equivalent Young's Modulus and Dutch cone bearing capacity (q_c) (kg/cm^2):

For footing length to width ratio (L/B):

1	$E_s = 2.5q_c$	(12)
10	$E_s = 3.5q_c$	(13)
1<L/B<10	interpolate between $2.5q_c$ and $3.5q_c$	(14)

If only SPT results are available to engineer, the SPT blow count value needs to be converted to CPT cone tip resistance value by using the q_c/N ratio.

In Situ Cone Penetration Test (CPT)

1. Schmertmann (1970) (cont'd.)

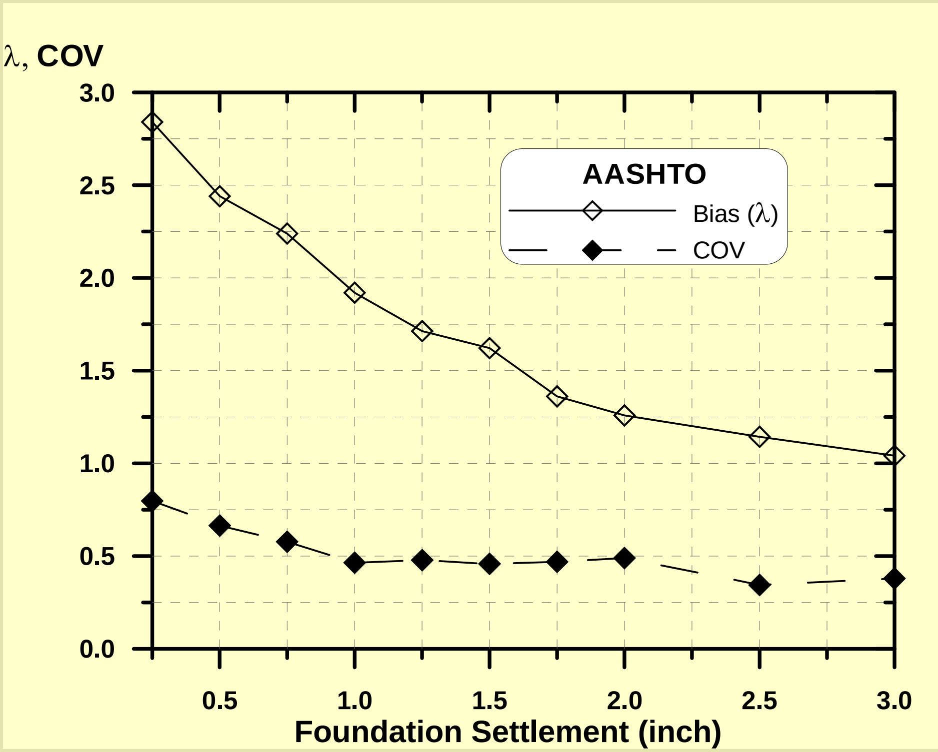
Table 4. Empirical Relationship of Modulus of Elasticity Empirical Ratio for SPT Converting, Schmertmann (1970)

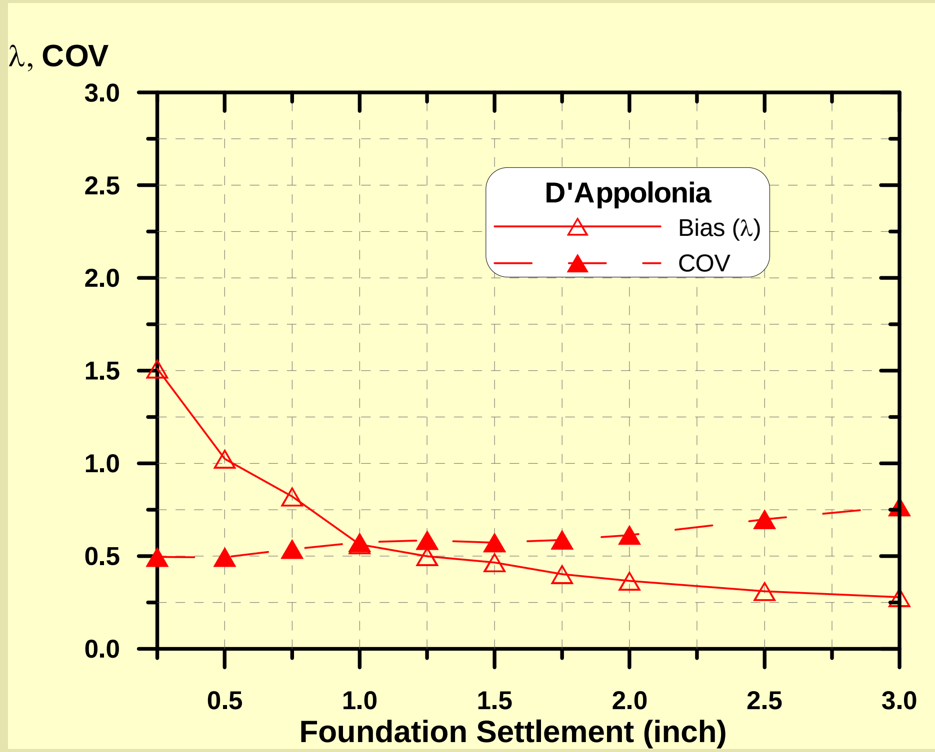
Empirical Equation		Reference
$E_s = 2.0q_c$	(E_s in tsf)	Schmertmann (1970)
$E_s = 2.5q_c$	(for axisymmetric cases, E in tsf)	Schmertmann & Hartman (1978)
$E_s = 3.5q_c$	(for plain strain cases, E in tsf)	
Empirical Value		
Soil Type	**q_c/N**	Schmertmann (1970)
Silts, sandy silts, slightly cohesion silt-sand mixtures	2	
Clean, fine to medium, sand & slightly silty sands	3.5	
Coarse sands & sand with little gravel	5	
Sandy gravels with gravel	8	

Notes: N = stand penetration resistance
E_s = modulus of elasticity
q_c = cone resistance

UNCERTAINTY OF THE SETTLEMENT ANALYSIS METHODS FOR FOOTINGS ON COHESSIONLESS SOIL

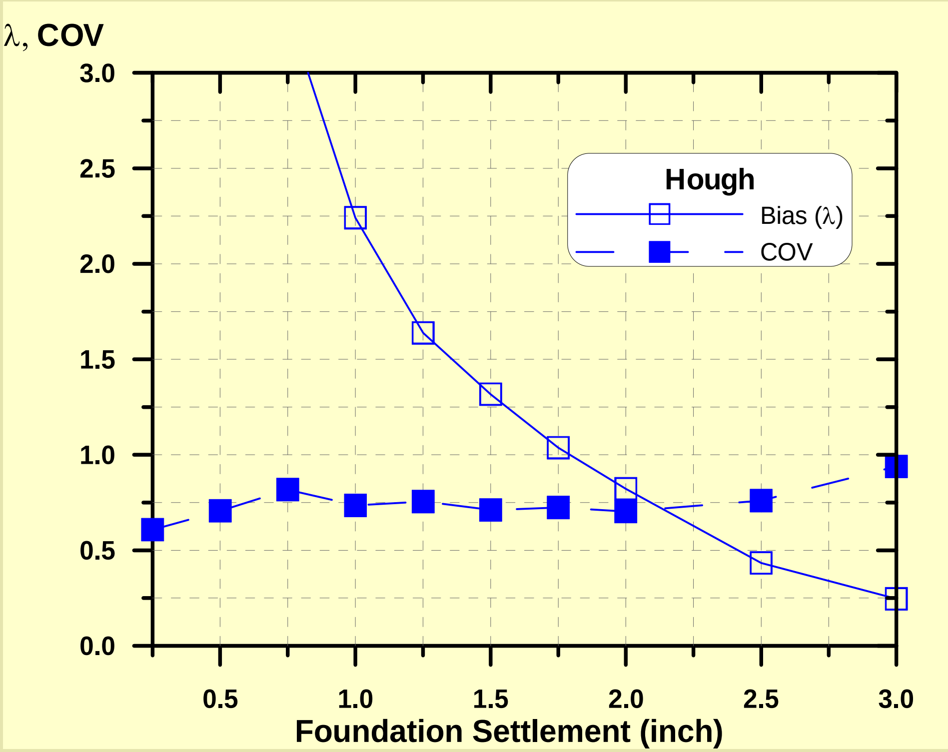
Shallow Foundations Bias & COV vs. Settlement

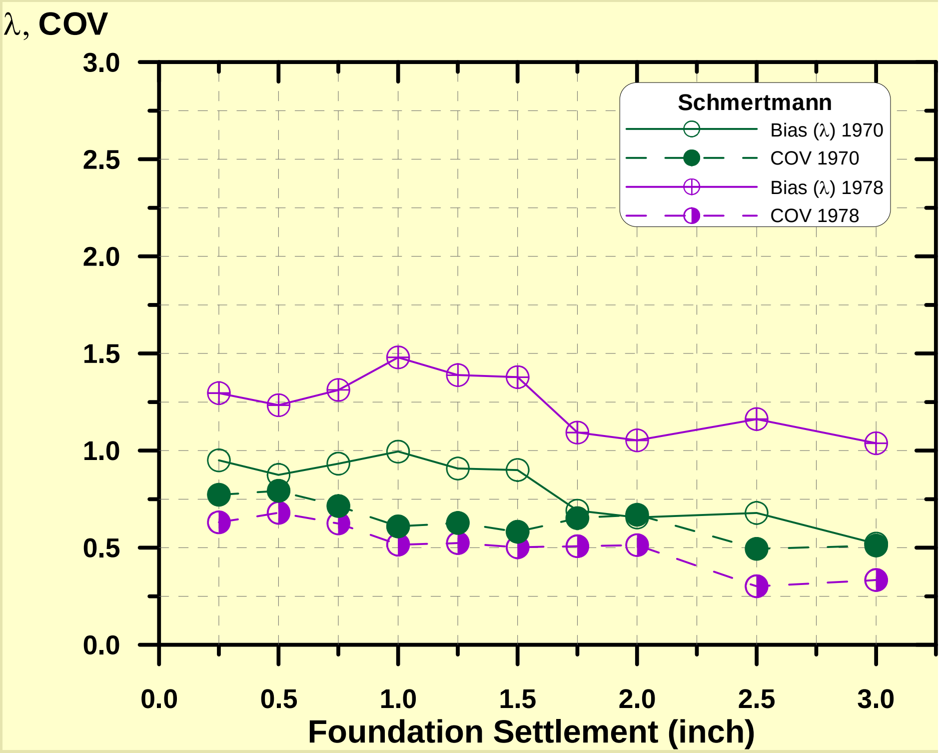




Bias (λ) Relates to the mean of the ratio of measured over calculated loads for a given displacement

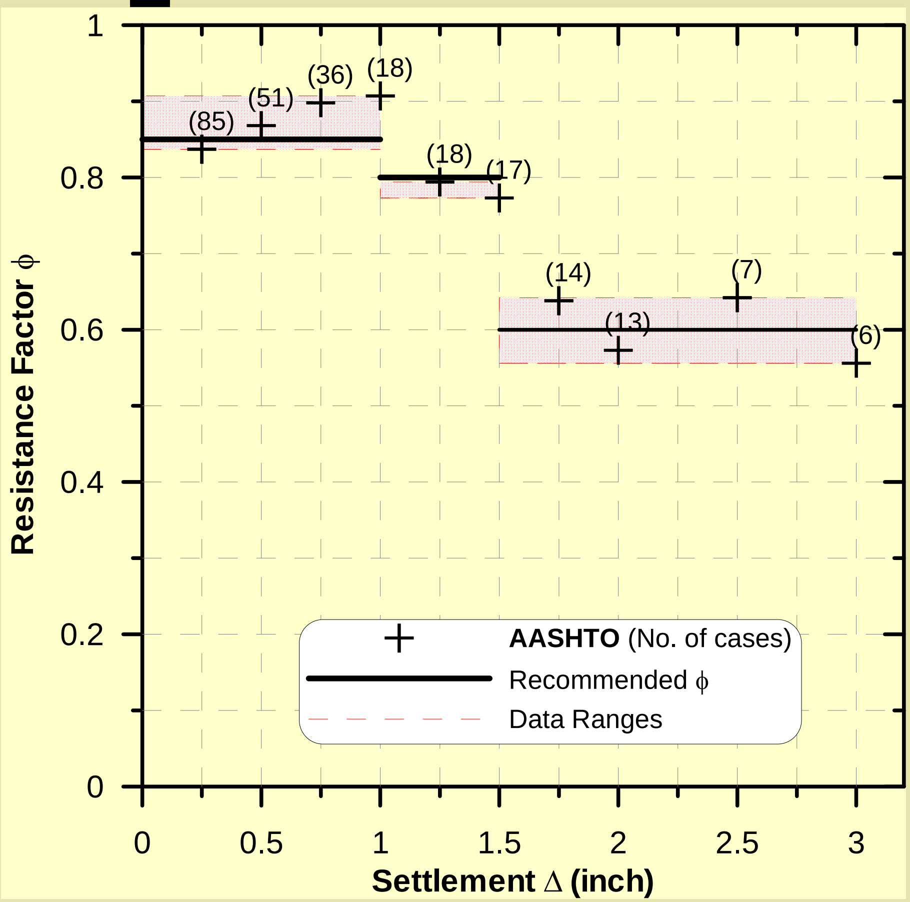
Shallow Foundations
Bias & COV vs. Settlement

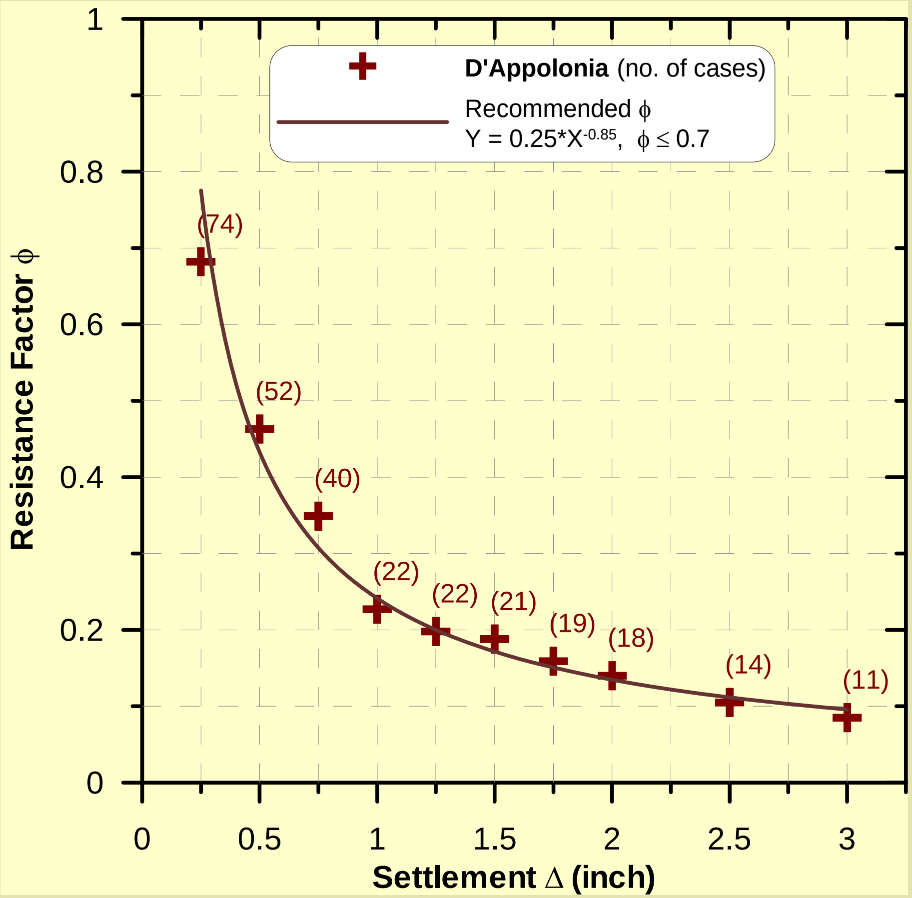




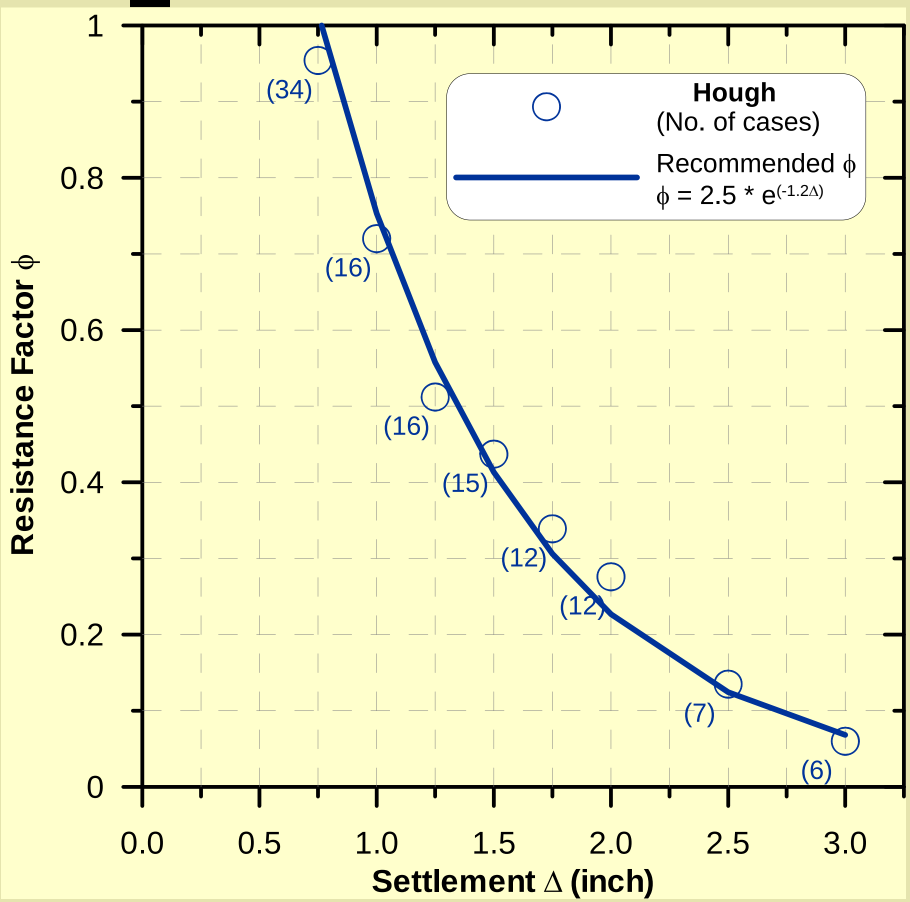
Bias (λ) Relates to the mean of the ratio of measured over calculated loads for a given displacement

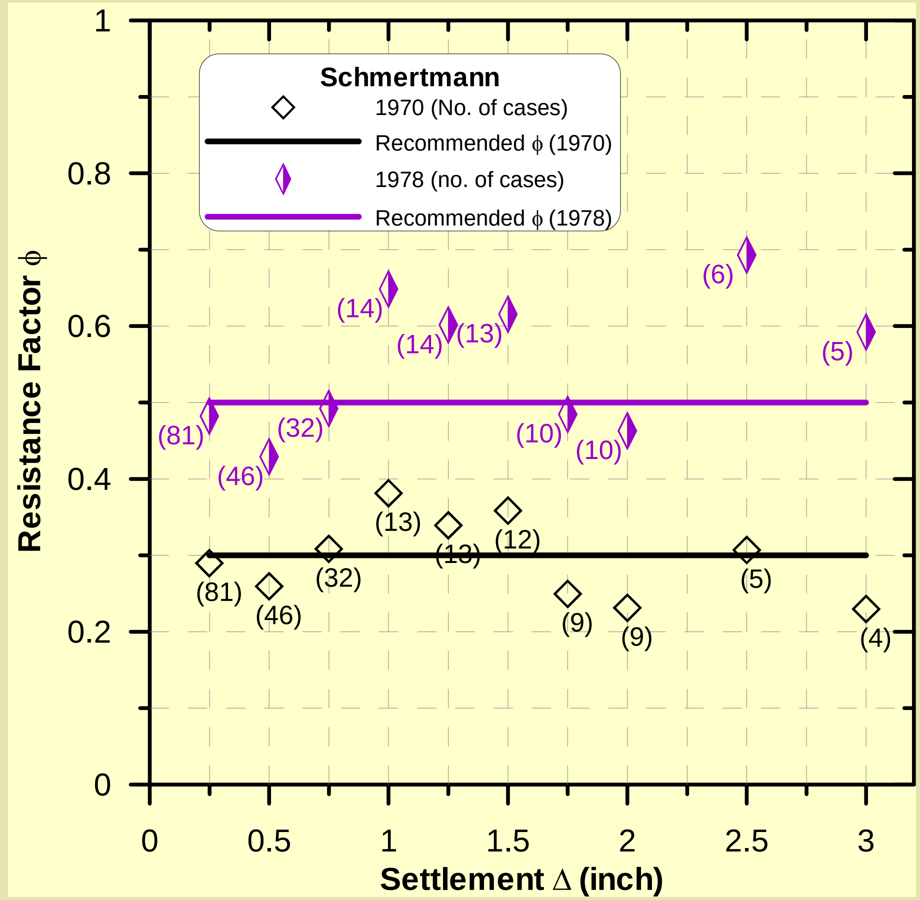
Shallow Foundations
Settlement vs. Resistance Factor φ





Shallow Foundations Settlement vs. Resistance Factor ϕ





Shallow Foundations
Settlement vs. Resistance Factor ϕ

Method		Range of Settlement Δ (inch)	n	λ	COV	ϕ mean	ϕ Recommended
AASHTO		0.25 – 1.0	190	2.532	0.734	0.878	0.85
		1.25 – 1.5	35	1.67	0.463	0.784	0.80
		1.75 – 3.0	40	1.24	0.448	0.602	0.60
Schmertmann	1978	0.00 – 3.00	231	1.279	0.599	0.550	0.50
	1970	0.00 – 3.00	224	0.894	0.734	0.295	0.30

Notes: n = number of measurements λ = bias (measured load over calculated load)
COV = coefficient of variation Δ = settlement ϕ = resistance factor

Shallow Foundations
Settlement vs. Resistance Factor ϕ

Method		Range of Settlement Δ (inch)	Resistance Factor ϕ
AASHTO		$0.00 < \Delta \leq 1.00$	0.85
		$1.00 < \Delta \leq 1.50$	0.80
		$1.50 < \Delta \leq 3.00$	0.60
Hough		$0.75 < \Delta \leq 3.00$	see figure or use equation $\phi=2.5e^{(-1.2\Delta)}$
Schmertmann	**1978**	$0.00 < \Delta \leq 3.00$	0.50
	1970	$0.00 < \Delta \leq 3.00$	0.30
D'Appolonia		$0.25 \leq \Delta \leq 3.00$	see figure or use equation $\phi=0.25\Delta^{(-0.85)}$ where $\phi \leq 0.7$

Notes: n = number of measurements λ = bias (measured load over calculated load)
COV = coefficient of variation Δ = settlement ϕ = resistance factor

Implementation Shallow Foundations Design Considering serviceability (controlling Criterion)

1. Evaluate B.C → Apply Ultimate Limit State Resistance Factor (Design Load x ϕ_{uls}) → Find Foundation Size

2. Determine Limit Settlement based on Serviceability Criterion

3. Calculate Foundation Size → the Applied Load is the Design Load x $\phi_{service}$ and the Settlement equals to the limit Settlement

Implementation
Design of Piles Under Vertical Loads Considering Serviceability

1. Evaluate B.C → Apply Ultimate Limit State Resistance Factor (Design Load x ϕ_{uls}) → Find Foundation Size

2. Determine Limit Settlement based on Serviceability Criterion

3. Calculate Foundation Size → the Applied Load is the Design Load x $\phi_{service}$ and the Settlement equals to the limit Settlement

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The Research Team:

- Samuel G. Paikowsky, Mary Canniff, GTR, Inc.
- Ayhan Garbuz, Yu Fu, Roiy Guy Geotechnical Eng. Research Lab., University of Massachusetts – Lowell
- Zeidan Ashraf, Guy Levi, Wisam Mualem and Sam Frydman, Technion – Israel Institute of Technology, Structural Engineering and Construction Management area of the Civil Engineering Department
- Japan Team: Yusuke Honjo, Gifu University, Ikumasa Yoshida, Shuichi Suzuki, Hyoudou Junichi, TEPSCO, Tokyo, Masahiro Shirato, PWRI, Japan
- Susan Faraji, Faraji Consulting, Inc., Winchester, MA


Thank You for Your Attention