RELIABILITY-BASED DESIGN IN GEOTECHNICAL ENGINEERING

T. H. Wu

OTEC 2011

THE OHIO STATE UNIVERSITY
DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING AND GEODETIC SCIENCE

1. How did It Begin?

- Reliability-based-design (RBD)

- Load-resistance-factor design (LRFD)

  Taylor (1948), Hansen (1965), Lumb (1970), Freundenthal (1956), Johnson (1953), Wu (1967, 1970…), ASCE Comm. on Reliability (1970+), NRC, Geotechnical Board Workshop (1993) , NCHRP (1992)

- AASHTO Code (1996-2011)
- Georisk 2011

2. What is LRFD?

- Design that accounts for uncertainties in model, load, and resistance

- Safety factor F_s = Resistance/load = R/Q

  - R and Q are uncertain quantities with mean (m), standard deviation (s) and coefficient of variation (V) = s/m

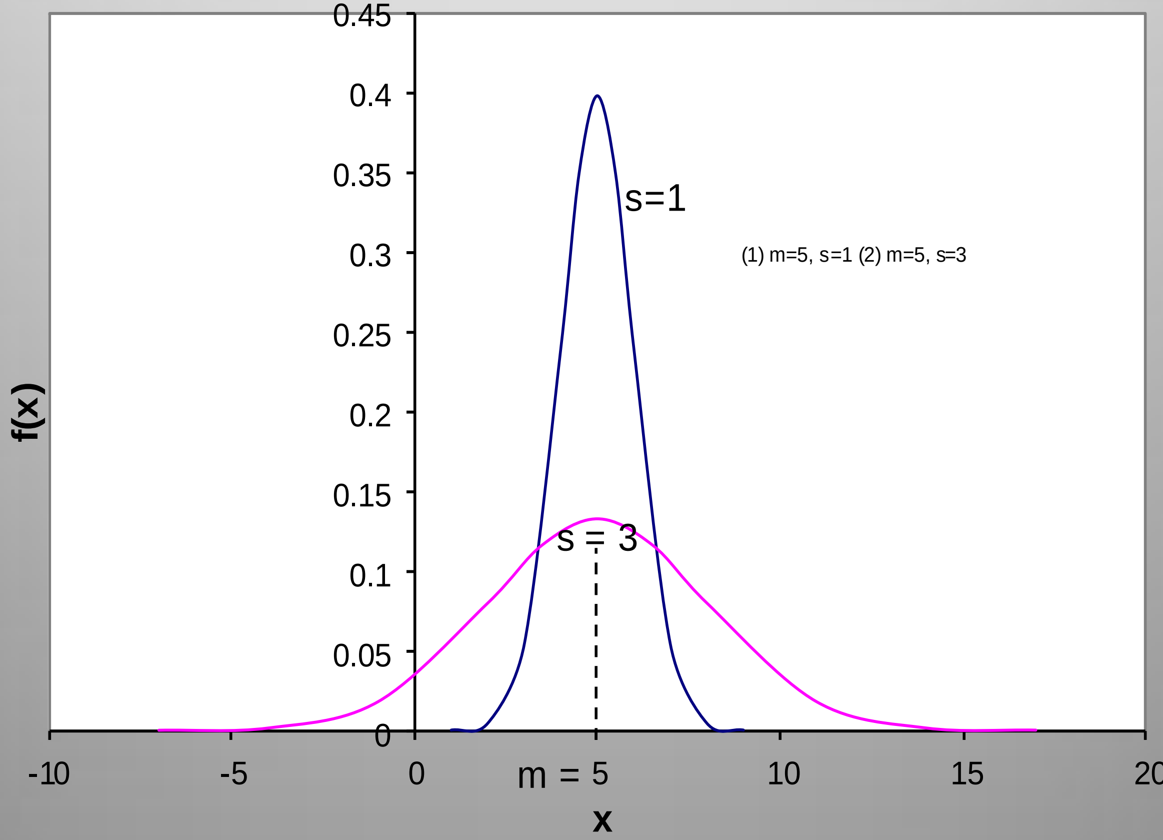
  - Choose F_s that gives an acceptable probability of failure P_f

THE OHIO STATE UNIVERSITY COLLEGE OF ENGINEERING

OTEC

3. Components of LRFD

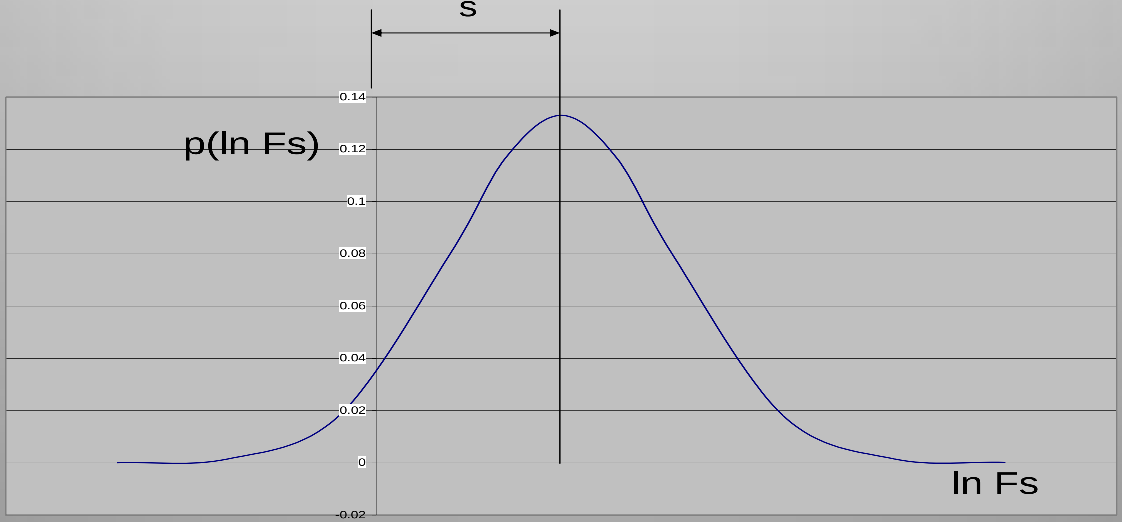
- Normal Probability Distribution
  - $f(x)$ = probability that x will fall between x_1 and $x_1 + dx$
  - $F(x) = 1-\beta$ = probability that x will be less than x_1
  - $\beta = (x_1 - m_x)/s$



4. Components of LRFD (continued)

- Probability of failure $P_f = 1- F_u [\beta]$
- For log-normal distribution

$$\beta = \frac{\ln\{\frac{\overline{R}}{\overline{Q}}[(1+V_Q^2)(1+V_R^2)]^{1/2}\}}{[\ln\{(1+V_Q^2)(1+V_R^2)\}]^{1/2}}$$



5. How to do LRFD (AASHTO) ?

- Require $F_s = R/Q > 1$, or $\gamma R_n / \gamma Q_n \geq 1$, or $\phi R_n \geq \gamma Q_n$
  - R_n = nominal resistance = resistance calculated according to some accepted procedure (e.g. Bearing capacity eq.)
  - Q_n = nominal load = load calculated according to some accepted procedure (ask the bridge engineer)
  - ϕ = resistance factor (AASHTO)

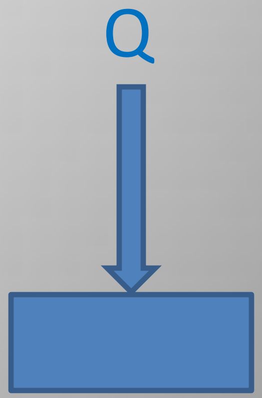
    ϕ is chosen to give a "target β" that conforms with F_s in current practice (calibration)
  - γ = load factor (AASHTO)

6. Example.

Bearing Capacity of Shallow Foundation on Sand

$q_{f} =$ 1000 kPa (Correlation with SPT)

D.L.= Q_1 = 2000 kN, L.L.= Q_2 = 1000 KN



$\gamma_1 = 1.25$ (Table 3.4.1-2), $\gamma_2 = 1.75$ (Table 3.4.1-1)

$$\gamma\, Q_n = \Sigma\, \gamma_i\, Q_i = 1[\,(2000 \times 1.25) + (1000 \times 1.75)\,]$$

$$= 4250 \text{ kN}$$

$\phi = 0.45$ (Table 10.5.5.2.2.1)

$$\phi\, R_n = 0.45 \times 1000\, A = 450\, A$$

$\gamma\, Q = \phi\, R_n$, $450\, A = 4250$ kN, $A = 9.4\ m^2$

Load Factors

Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts		1.95	0.90
• Flexible Metal Box Culverts and Structural Plate Culverts with Deep Corrugations		1.50	0.90
ES: Earth Surcharge		1.50	0.75

Load Factors

Table 3.4.1-1—Load Combinations and Load Factors

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time			
										EQ	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
Fatigue I—*LL*, *IM* & *CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—
Fatigue I II—*LL*, *IM* & *CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—

Resistance Factors

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

THE OHIO STATE UNIVERSITY COLLEGE OF ENGINEERING

OTEC

7. Some Important Details. What is ϕ?

$Q = \lambda_Q \; Q_n, \qquad R = \lambda_R \; R_n$

- λ_Q , λ_R = bias in prediction model
- β by calibration with current practice (Rojiani et al, 1991) (Table 2; Table 7.2, 7.3 in Barker et al, 1991)

$$\phi = \frac{\lambda_R \gamma_Q Q_n [(1+V_Q^2)(1+V_R^2)]^{1/2}}{\lambda_Q Q_n \exp\{\beta[\ln(1+V_Q^2)(1+V_R^2)]^{1/2}\}}$$

THE OHIO STATE UNIVERSITY COLLEGE OF ENGINEERING

OTEC

8. How did they get the numbers ?

Summary of Statistics for Ultimate Bearing Capacity of Footings on Clay

Parameter	Bias	Cov	Remarks/Reference
Model Error	1.01	0.045	Skempton's data (1951)
Soil Parameters S_u (lab test) S_u (field vane) S_u (from q_c)	 1.00 0.95 1.00	 0.11 0.15 0.18	 Singh and Lee (1974) Wu and Lee (1988) Lunne, et al. (1983) Hoeg & Tang (1977)
Inherent spatial variability	1.0	$\delta_z V_{pt} / \sqrt{2B}$	Baecher, et al. (1983)

Table 6-6
Approximate Guidelines for Design Property Variability
(after Phoon, et al., 1995)

Design Property	Test	Soil Type	Point COV (%)	Spatial Avg. COV (%)
S_u (UC)	Direct (lab)	Clay	20-55	10-40
S_u (UU)	Direct (lab)	Clay	10-35	7-25
S_u (CIUC)	Direct (lab)	Clay	20-45	10-30
S_u (field)	VST	Clay	15-50	15-50
S_u (UU)	q_T	Clay	30-40	30-35
S_u (CIUC)	q_T	Clay	35-50	35-40
S_u (UU)	N	Clay	40-60	40-55
S_u (field)	PI	Clay	30-55	--
ϕ_f'	Direct (lab)	Clay, Sand	7-20	6-20
ϕ_f' (TC)	q_T	Sand	10-15	10
E_{PMT}	Direct (PMT)	Sand	20-70	15-70
E_{PMT}	N	Clay	85-95	85-95
E_D	N	Silt	40-60	35-55

S_u = Undrained shear strength; UC = Unconfined compression test; UU = Unconsolidated-undrained triaxial compression test; CIUC = Consolidated isotropic undrained triaxial compression test; S_u (field) = Corrected S_u from vane shear test; ϕ_f' = Effective stress friction angle; TC = Triaxial compression test; q_T = Corrected cone tip resistance; E_{PMT} = Pressuremeter modulus; E_D = Dilatometer modulus averaged over a depth interval of 5 m

Example of Correlation

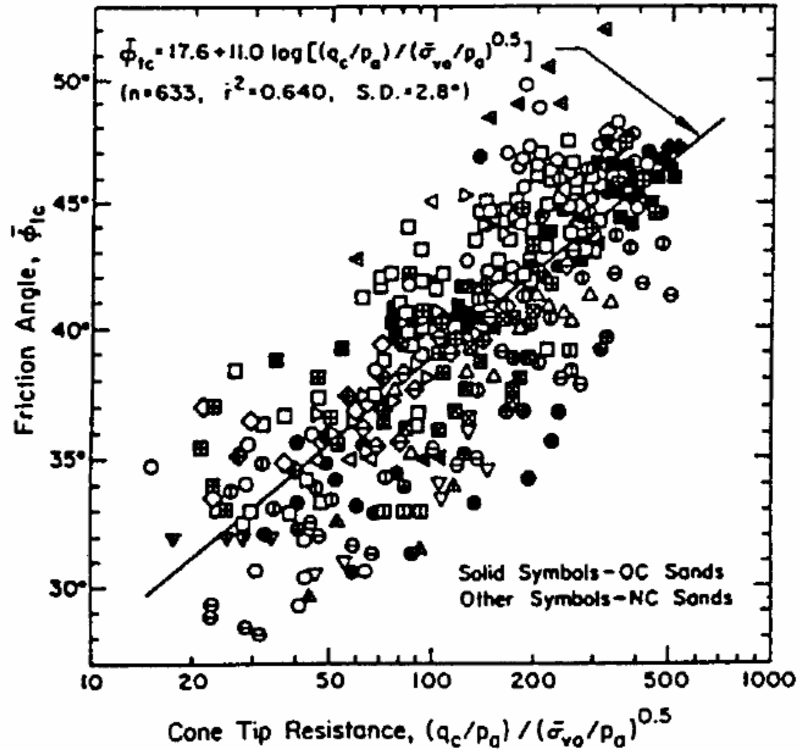


Figure 6-5

Correlation of Triaxial Compression Effective Stress Friction Angle with Cone Penetration Tip Resistance

(after Kulhawy and Mayne, 1990)

9. Words of Wisdom

“Local experience, local geologic formation specific property correlations, and knowledge of local geology, in addition to broader based experience and relevant published data , should be considered in the final selection of design parameters” (AASHTO, 2006, p. 10-13)

“Regionally specific values should be determined based on substantial statistical data combined with calibration or substantial successful experience to justified higher values. Smaller resistance factors should be used if site or material variability is anticipated to be unusually high …” (AASHTO, 2006, p. 10-33)

THE OHIO STATE UNIVERSITY COLLEGE OF ENGINEERING

OTEC

10.Some Questions. What is next?

- What are “local …” and “regionally specific values” for Ohio soils?

  - Variability of typical Ohio soils (glacial deposits, residual soils, …) → V_R

  - Correlations of strength with test data (CPT, SPT ..) → λ_R and V_R

  - Others?

11. Limitations of LRFD

LRFD cannot account for human errors (bad judgment). The most common cause of geotechnical failures is misjudgment of site conditions.

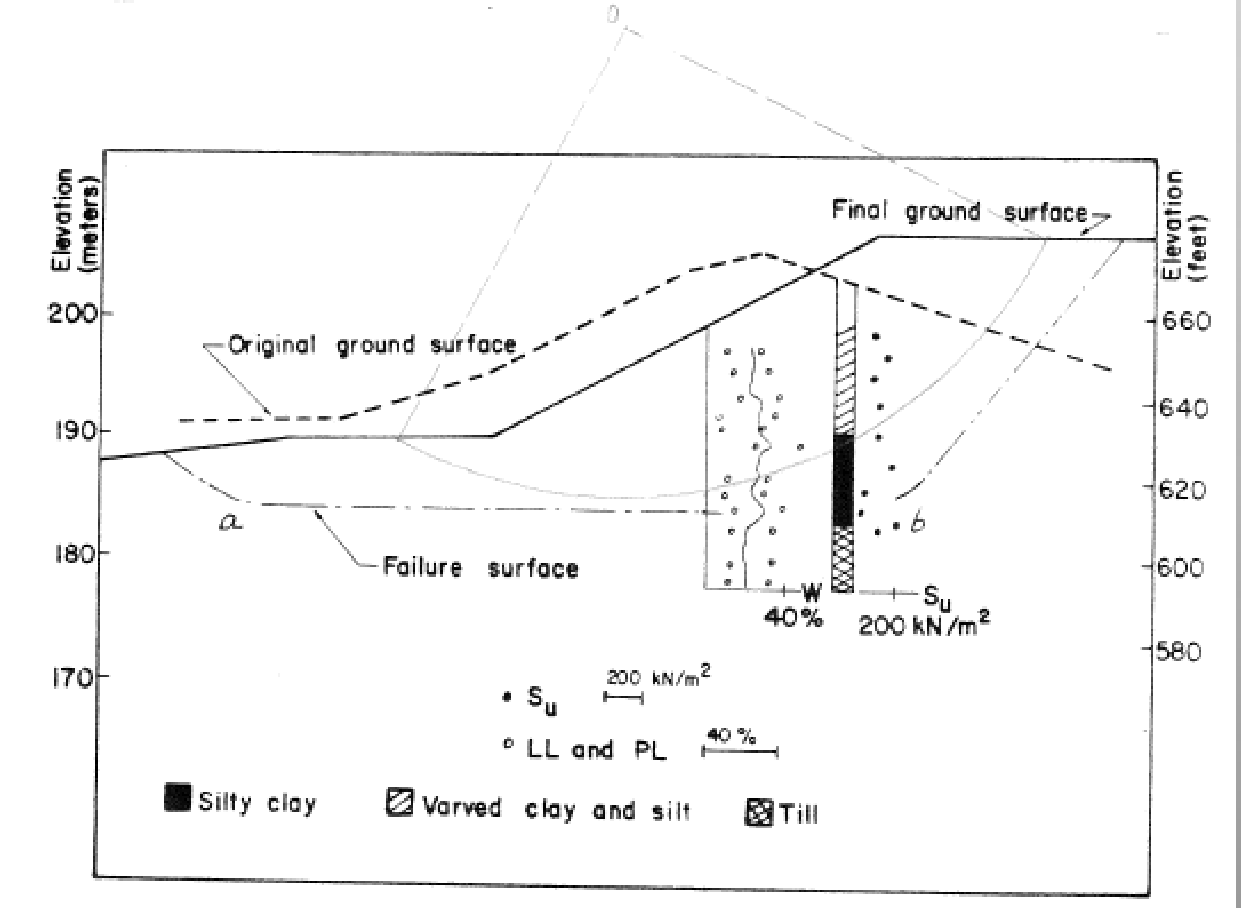


Figure 2: Section A-A, Preliminary Information

12. More Words of Wisdom.

"A natural soil is never homogeneous. Its properties change from point to point, while our knowledge of these properties is limited to those few spots at which the samples have been collected”. Terzaghi (Presidential Address, *Proc. 1st International Conf.,* 1936)

“Nature can outwit us and does”. Peck (ASCE Specialty Conf., 1972)