Development of LRFD Resistance Factors for External Stability of MSE Walls

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Scope of Project

- Determine LRFD resistance factors (Φ) for external stability (sliding and bearing) of MSE walls through centrifuge tests and analytical techniques

- Recommendations of Φ are to be made in the form of equations or tables for L/H and summary statistics of soil variability (i.e., μ_ϕ, CV_ϕ, μ_γ, CV_γ)

Project Overview

- Task 1: Determine design methods, typical wall geometries, typical soil conditions, design parameters
- Task 2: Make sensitivity assessments of external stabilities for wall design parameters and soil variability
- Task 3: Perform centrifuge tests of model walls sliding and bearing stabilities with soil variability
- Task 4: Assess centrifuge results and compare with conventional design methods
- Task 5: Develop LRFD resistance factors (Φ) from Task 4 results for MSE wall modes of external stability
- Supplement Task 1: Centrifuge tests of MSE wall on/near embankments with soil variability
- Supplement Task 2: Simulations establishing probability of failures using existing bearing capacity or overall stability methods
- Supplement Task 3: Compare results from Task 1 and 2, make adjustments to methods and recommend LRFD resistance factors (Φ) in form of equations or charts

MSE Walls-Current Practice

- Most wall heights 20 ft – 40 ft
- Typically L/H = 0.7 – 1.2
- Horizontal and sloping backfills
- Rankine and Coulomb analysis
- AASHTO recommends use of Vesic's N_γ and i_γ
- Design surcharge, q_s, = 250 psf

  Suitable soils

  A-1-a (GW) and A-3 (SP)

  A-1-b (GM), A-2-4,5 (SM), A-2-6,7 (SC) <15% fines
- Observed variability in backfill densities and friction angles for MSE walls in Florida – $CV_\gamma \approx 5\%$, $CV_\phi \approx 10\%$

MSE Walls-Current AASHTO LRFD

- $\Phi R_n \geq \eta \Sigma\gamma_i Q_i$

  η = 1.0 - redundancy

  γ = load factor (dead, vertical, horizontal, surcharge, etc.)

  Q = load or force effect

  Φ = resistance factor

  R_n = nominal resistance (force)
- AASHTO recommended Φ values (external stability)

  Sliding: Φ = 1.0

  Bearing: Φ = 0.65

  Overall: Φ = 0.65-0.75

  No Φ for Overturning
- Note, Φ values calibrated to FS values-not a probabilistic approach
- The Φ values don't explicitly account for variability of soil or load inclination

Sensitivity Study

- Summary statistics of soil properties*

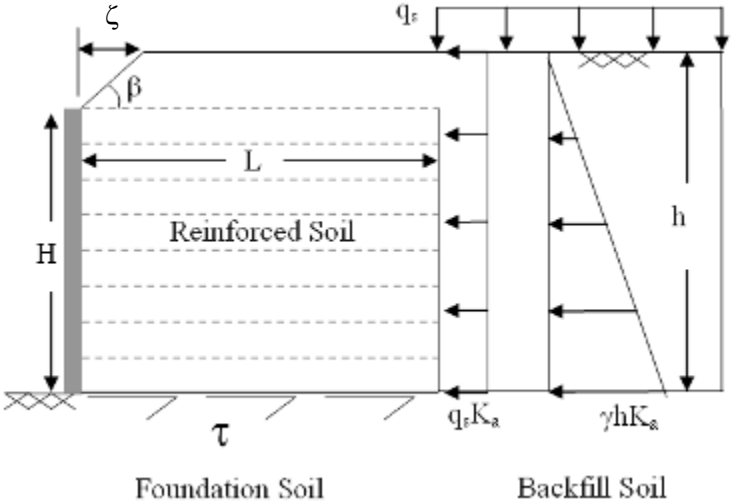
- Variable surcharge (q_s)

	Baseline Parameters		Range of Parameters		
	Mean (μ)	Coefficient of Variation (CV)	Mean (μ)	Coefficient of Variation (CV)	Distribution
Reinforced Soil ϕ	30°	10%	20° - 40°	5% - 20%	Lognormal
Reinforced Soil γ	105 pcf	5%	95 pcf - 120 pcf	5% - 20%	Lognormal
Backfill Soil ϕ	30°	10%	20° - 40°	5% - 20%	Lognormal
Backfill Soil γ	105 pcf	5%	95 pcf - 120 pcf	5% - 20%	Lognormal
Foundation Soil ϕ	35°	10%	20° - 40°	5% - 20%	Lognormal
Foundation Soil γ	105°	5%	95 pcf - 120 pcf	5% - 20%	Lognormal
Surcharge q_s	250 psf	25%	NA	NA	Lognormal

- Use Monte Carlo analysis for sensitivity study of parameters (e.g., μ_ϕ, CV_ϕ, μ_γ, CV_γ, L, etc.) and identify those that control for centrifuge tests

* Goh, 2009; Fenton and Griffiths, 2008; Duncan, 2000; Zevgolis and Bourdeau, 2006

Sliding Stability



μ_ϕ and CV_ϕ (σ_ϕ / μ_ϕ)

μ_γ and CV_γ ($\sigma_\gamma / \mu_\gamma$)

$$CDR_{sliding} = \frac{F_{resisting}}{F_{driving}} = \frac{(L\gamma h Tan(\phi)\alpha_{EV}) \bullet \Phi}{1/2\gamma h^2 K_a \alpha_{EH} + q_s h K_a \alpha_{LS}}$$

where

L = reinforcement length

h = wall height

γ = soil unit weight

ϕ = foundation soil friction angle

K_a = active earth pressure coefficient (Rankine, Coulomb)

q_s = uniform surcharge

α_{EV}, α_{EH}, α_{LS} = load factors for vertical earth load, horizontal earth load and surcharge load

Φ = resistance factor

note: if $\beta = 0$, $h = H$

<u>Sensitivity Analysis</u>

- Greatest influence on $P_{f_{sliding}}$ from:
  - CV_ϕ and μ_ϕ of backfill and foundation soil;
  - CV_γ backfill
- Least influence from backfill mean unit weight
- Influence of variable surcharge (CV = 25%), q_s, is negligible

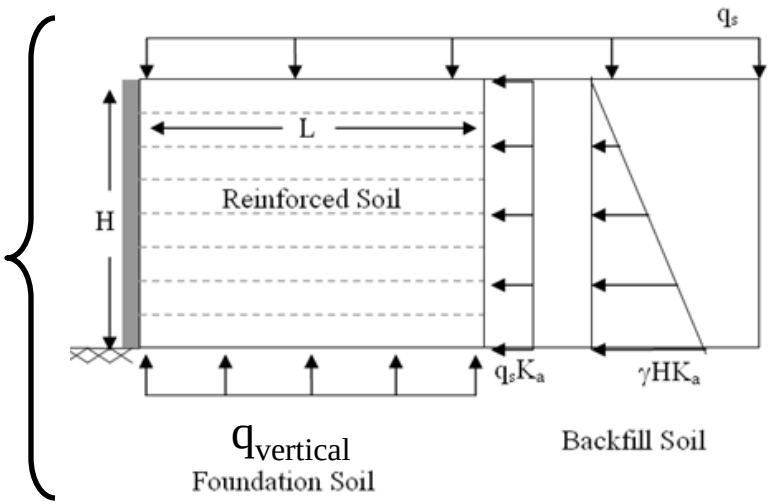
Bearing Stability

$$CDR_{bearing} = \frac{q_{ultimate}}{q_{vertical}} = \frac{(0.5\gamma L N_\gamma) \bullet \Phi}{\dfrac{(\gamma H L \alpha_{EV} + q_s L \alpha_{LS})}{L'}}$$

$$L' = L - 2e$$

μ_ϕ and CV_ϕ (σ_ϕ / μ_ϕ)

μ_γ and CV_γ ($\sigma_\gamma / \mu_\gamma$)

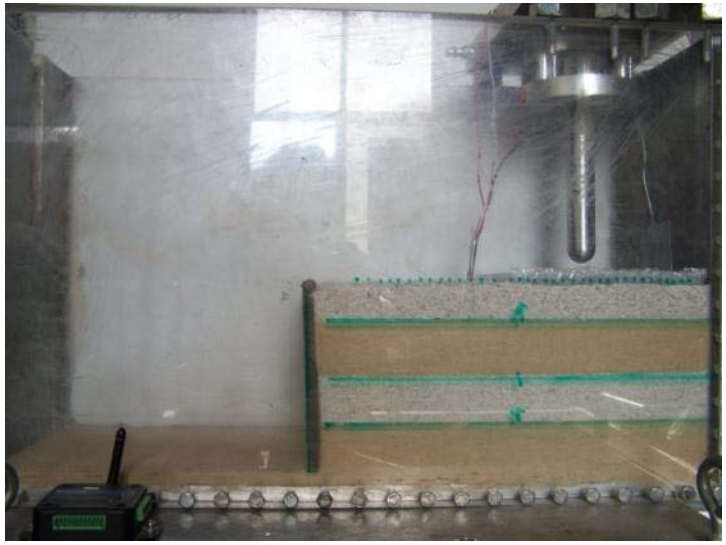


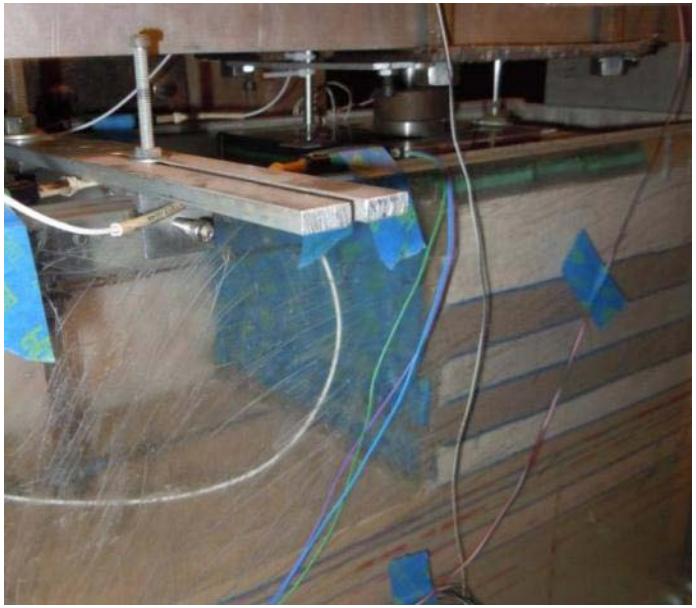
where
L = reinforcement length
L' = effective foundation length
e = eccentricity
H = wall height
γ = soil unit weight
N_γ = unit weight factor
q_s = uniform surcharge
K_a = active earth pressure coefficient (Rankine, Coulomb)
α_{EV}, α_{LS} = load factors for vertical earth load and surcharge load
Φ = resistance factor

<u>Sensitivity Analysis</u>

- Greatest influence on $P_{f_{bearing}}$ from foundation soil's CV_ϕ and μ_ϕ
- Some influence from backfill mean unit weight and mean friction angle
- Some influence from foundation soil's mean unit weight
- Influence of variable surcharge (CV = 25%), q_s, is negligible

Centrifuge Tests



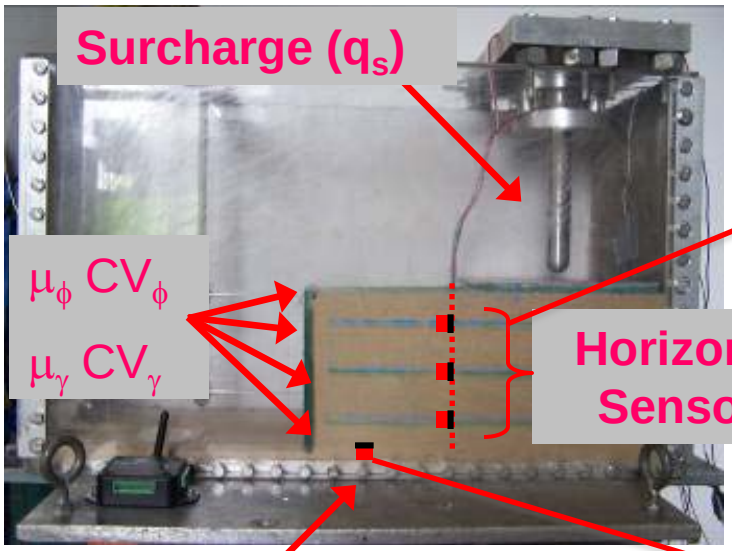


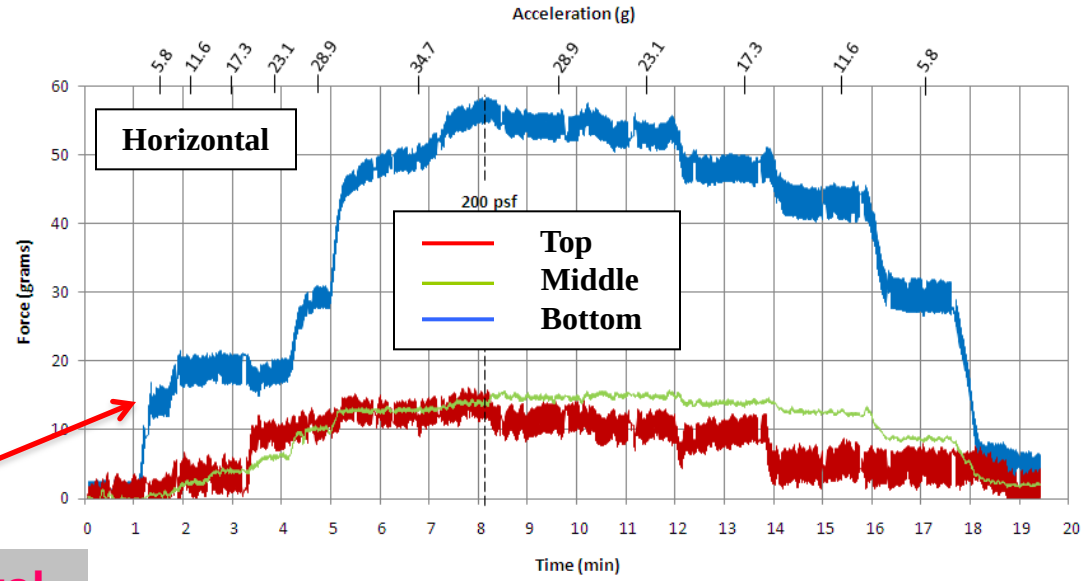
- Flexible wall facing and carbon steel reinforcement
- Wall constructed at 1-G in segments
- Internal stabilities for sliding and bearing: $CDR_{pullout} > 2.0$ and $CDR_{breaking} > 2.0$
- Lateral and vertical displacements and stresses were monitored
- Surcharge load monitored with load cell

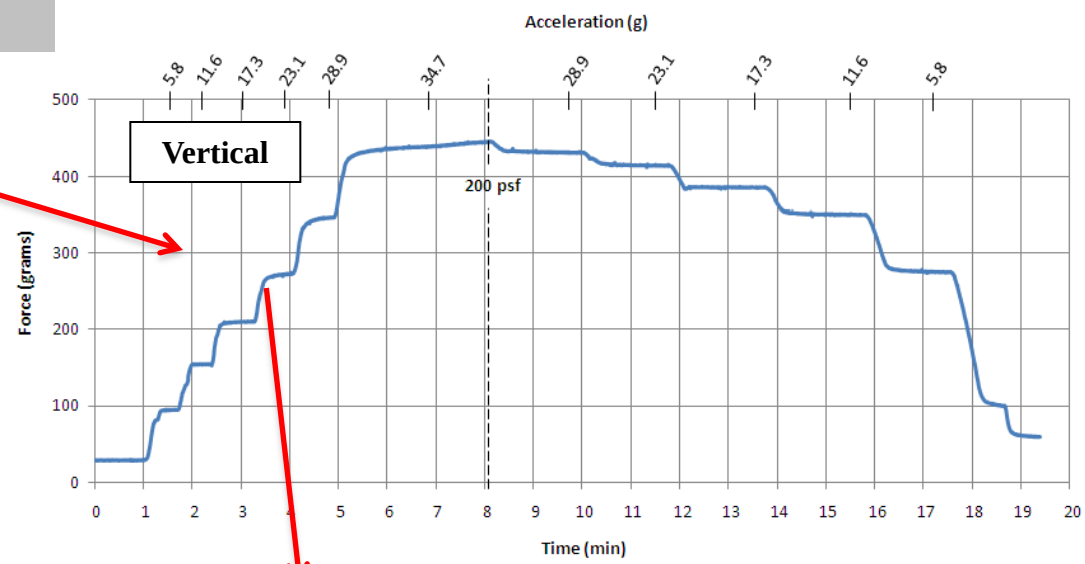
Assessing and Modeling Soil Variability

- Since the foundations soil's friction angle is controlling, low to high strength soils in the dense state (D_r >60%) were needed
- Multiple soils (A5 Vicksburg silt to A3 Ottawa sand to A-2-4 slightly silty find sand) were obtained and tested for D_{10}, D_{60}, D_r and ϕ
- Soil was pluviated into layers then surface was vacuumed to obtain uniform thickness and unit weight
- Each layer is uniformly compacted
- Density determined by mass and volume measurements before test
- Friction angle-density relationship of each test soil determined with the direct shear test performed at different densities and for normal stresses, σ_N, up to 75 psi
- CV_ϕ assessed based on measured values

Sliding Model Tests



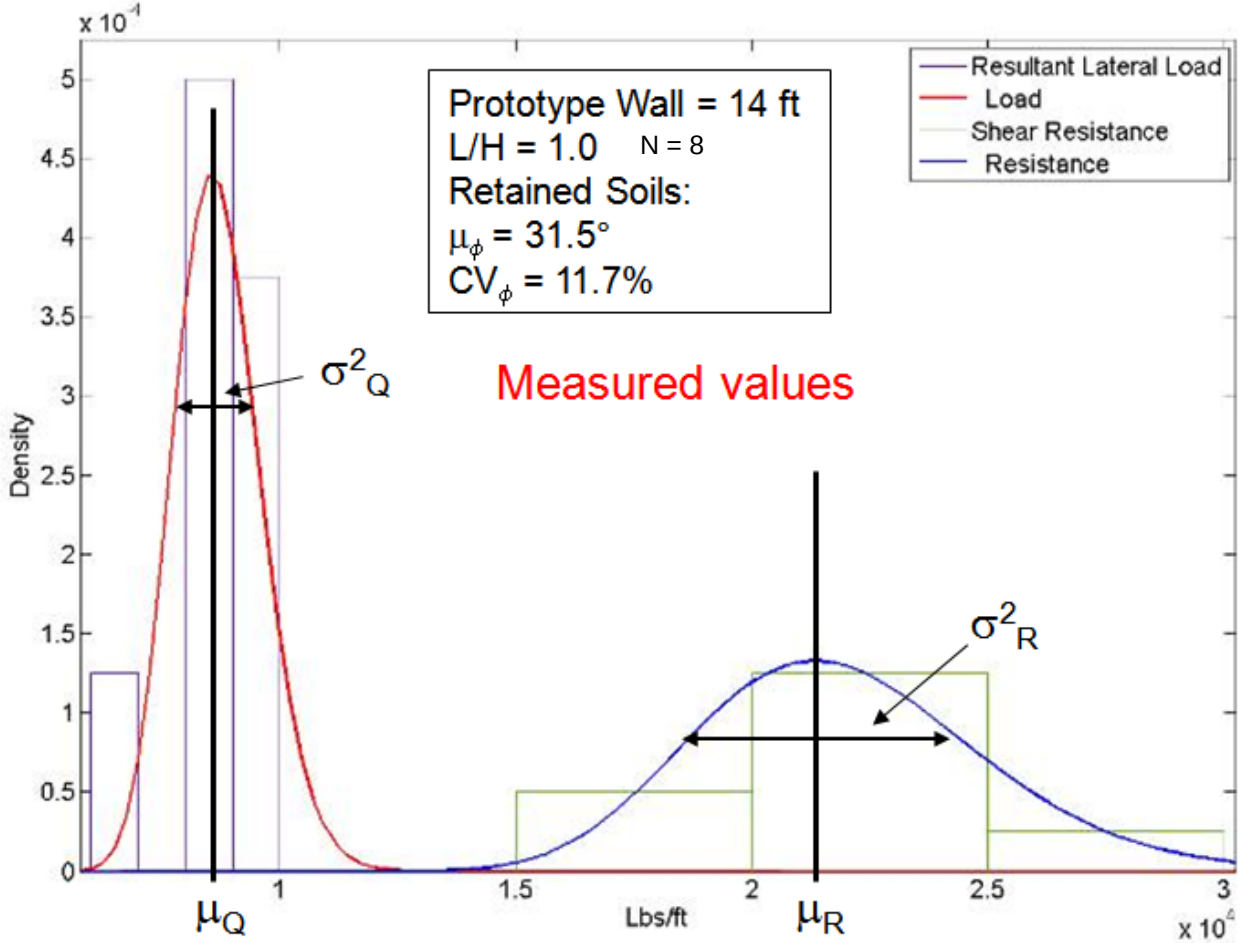




Acceleration (g)	Prototype Wall Height (ft)
34.67	17.57
28.90	14.64
23.11	11.71
17.34	8.79
11.56	5.86
5.78	2.93

$$\tau = \sigma_N \tan(\phi)$$

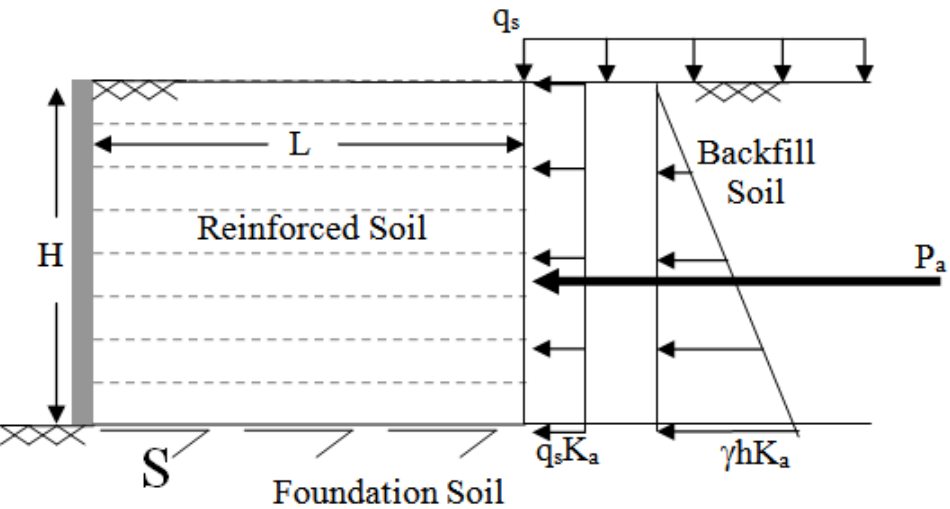
Sliding Test Results – Load and Resistance



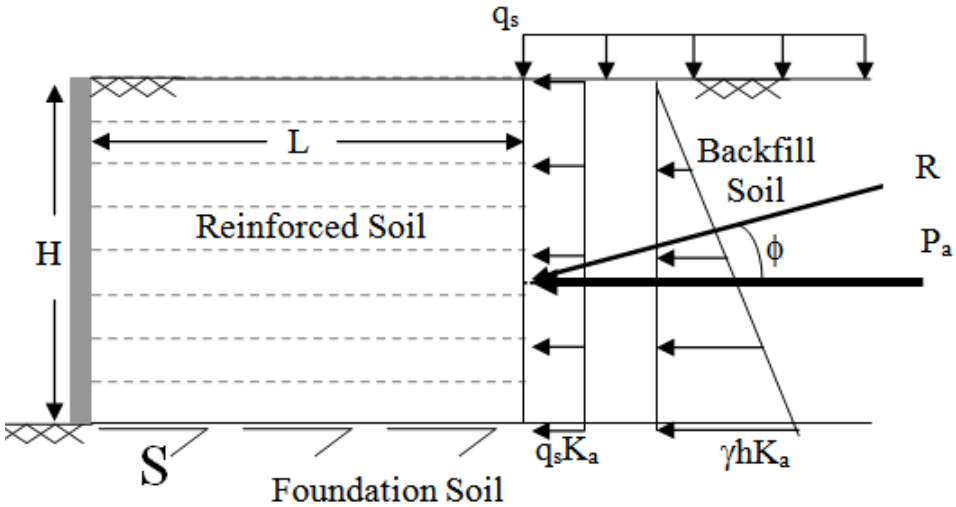
$$CV_Q = \frac{\sqrt{\sigma^2_Q}}{\mu_Q}$$ and $$CV_R = \frac{\sqrt{\sigma^2_R}}{\mu_R}$$ → Applied in LRFD $\Phi_{sliding}$

Methods of Sliding for LRFD

Rankine



Coulomb



$$P_a = \frac{1}{2}\gamma_{bf}H^2K_a + q_sHK_a$$

$$S = \gamma_{rs}HL\tan(\phi_{fs})$$

$$R = \frac{1}{2}\gamma_{bf}H^2K_a + q_sHK_a$$

$$P_a = R\sin(\phi)$$

$$S = \gamma_{rs}HL\tan(\phi_{fs})$$

where

L = reinforcement length, H = wall height, ϕ_{fs} = foundation soil friction angle,
q_s = uniform surcharge, K_a = active earth pressure coefficient (Rankine, Coulomb),
γ_{bf} = backfill soil unit weight, γ_{fs} = foundation soil unit weight, ϕ = backfill soil friction angle

Sliding LRFD Φ

L/H =1 tests with backfill μ_ϕ = 31.5° and CV_ϕ = 11.7%

Rankine

Wall Height (ft)	No. Values (n)	Measured Resistance (S) μ_R (lb/ft)	CV_R	Measured Load (P_a) μ_Q (lb/ft)	CV_Q	Mean Bias = (Measured/Predicted) λ_R	λ_D	λ_L	Load Factors (AASHTO) α_{EV}	α_{EH}	α_{LS}	Φ
8.3	10	2891.90	0.81	1908.10	0.80	1.21	1.4	1.4	1	1.5	1.75	0.79
11.1	10	4338.90	0.80	2799.63	0.81	1.04	1.2	1.2	1	1.5	1.75	0.94
13.8	8	4671.10	0.76	2504.57	0.16	0.6	0.7	0.7	1	1.5	1.75	0.74

Coulomb

Wall Height (ft)	No. Values (n)	Measured Resistance (S) μ_R (lb/ft)	CV_R	Measured Load (P_a) μ_Q (lb/ft)	CV_Q	Mean Bias = (Measured/Predicted) λ_R	λ_D	λ_L	Load Factors (AASHTO) α_{EV}	α_{EH}	α_{LS}	Φ
8.3	10	2891.90	0.81	1908.10	0.80	1.21	1.8	1.8	1	1.5	1.75	0.62
11.1	10	4338.90	0.80	2799.63	0.81	1.04	1.5	1.5	1	1.5	1.75	0.63
13.8	8	4671.10	0.76	2504.57	0.16	0.6	0.8	0.8	1	1.5	1.75	0.67

λ_R: Mean bias in the resistance

λ_D: Mean bias in the dead load, uses Rankine's or Coulomb's theory

λ_L: Mean bias in the live load, here was taken to be equal to the mean bias in dead load

α_{EV}: Load factor for vertical earth loads

α_{EH}: Load factor for horizontal earth loads

α_{LS}: Load factor for surcharge loads

<u>UF values</u>

Rankine: Φ = 0.74 – 0.94

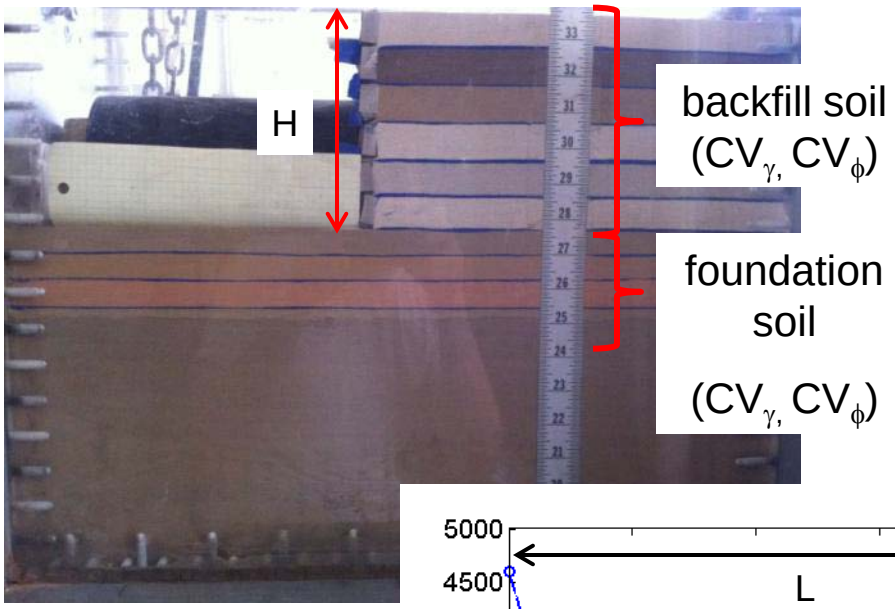
Coulomb: Φ = 0.62 – 0.67

<u>AASHTO value</u>

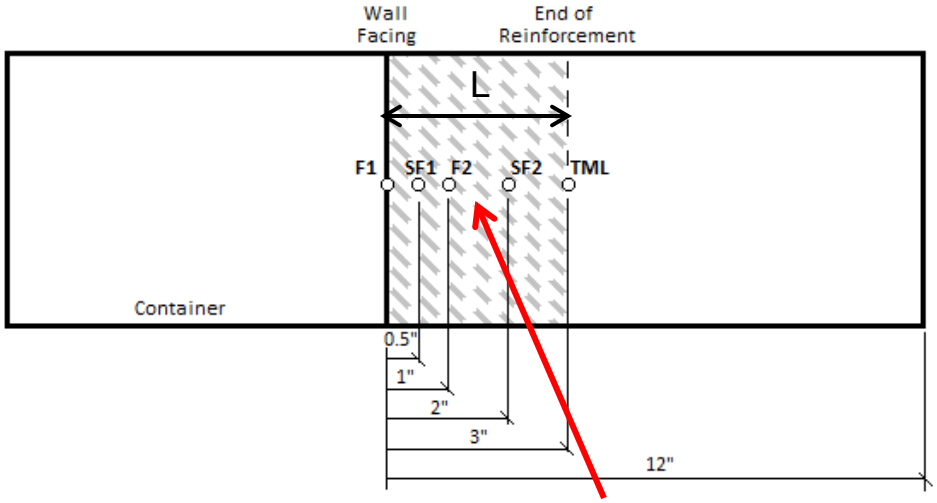
Φ = 1.0

Bearing Capacity Tests

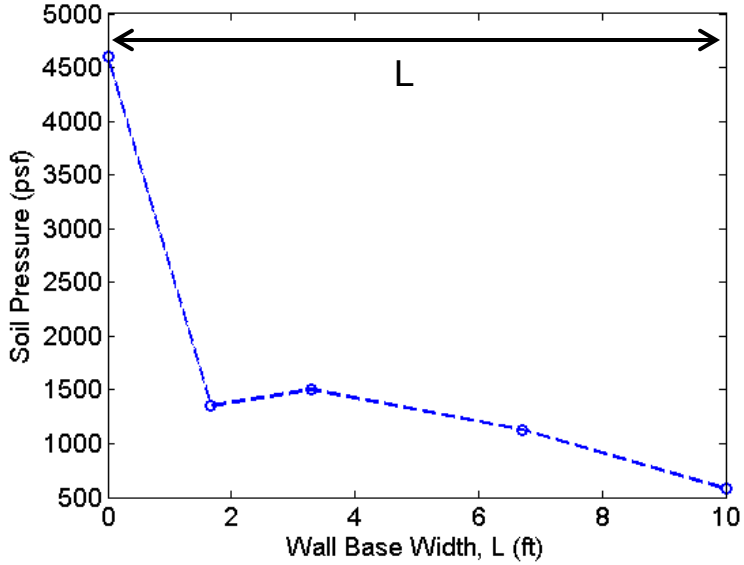
Side View



Plan View



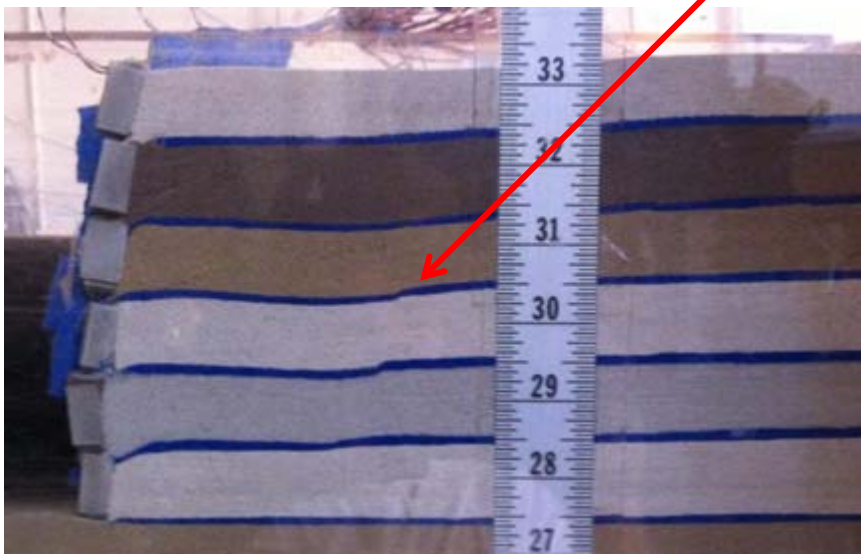
Miniature soil pressure sensors placed along center

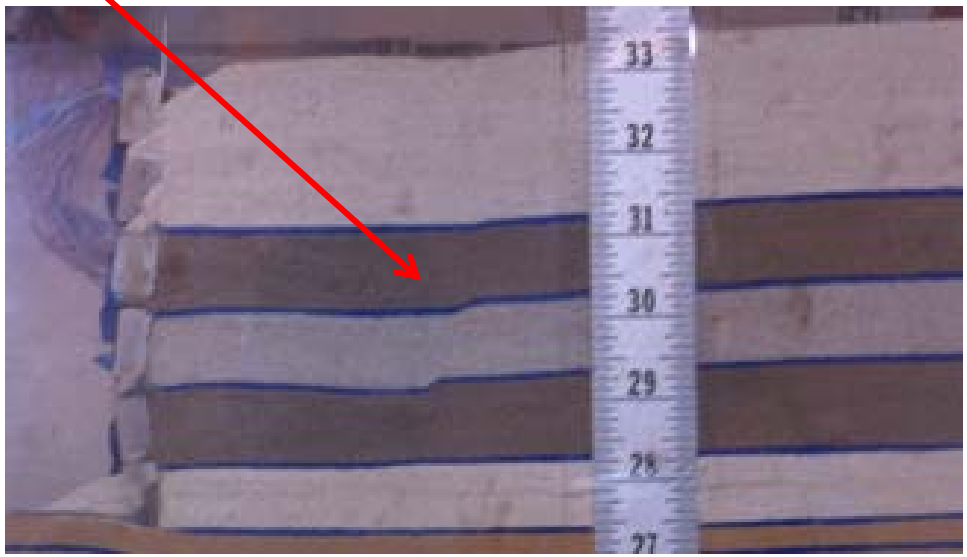


Measured soil pressure distribution

Bearing Capacity Tests

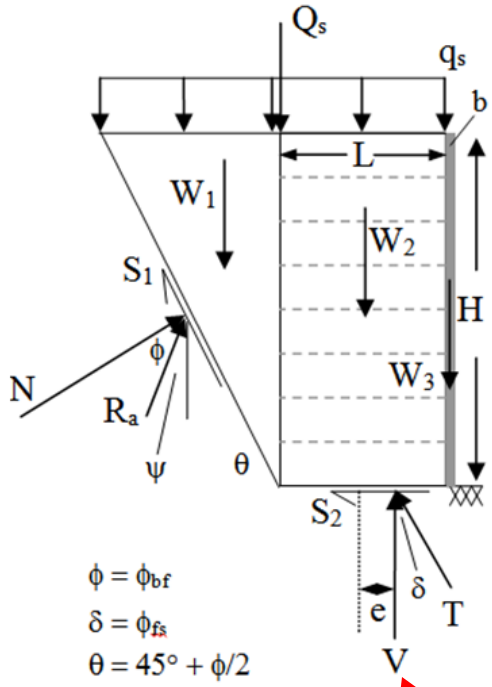
Observed rupture surface in backfill

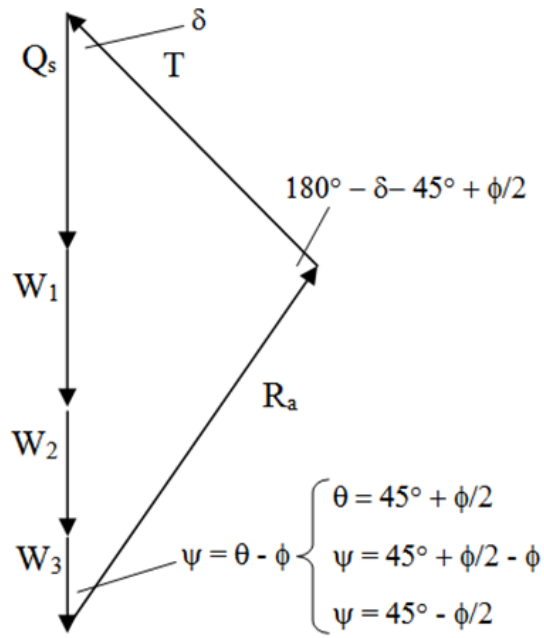




Bearing Capacity Tests

- Failure of wall can be described by a planar rupture surface through backfill (observed in tests)
- Vertical resultant force, V, can be calculated based on force equilibrium



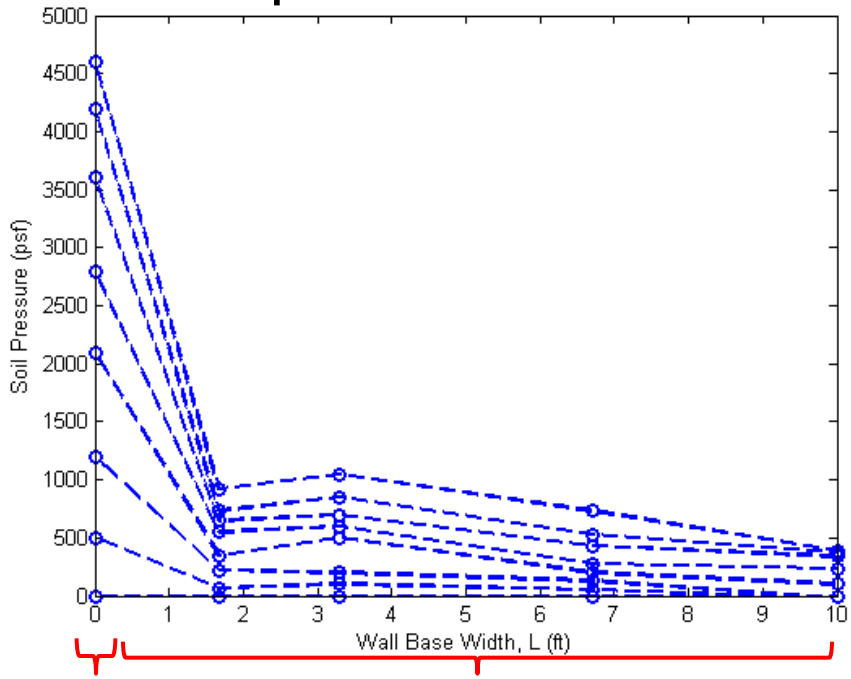


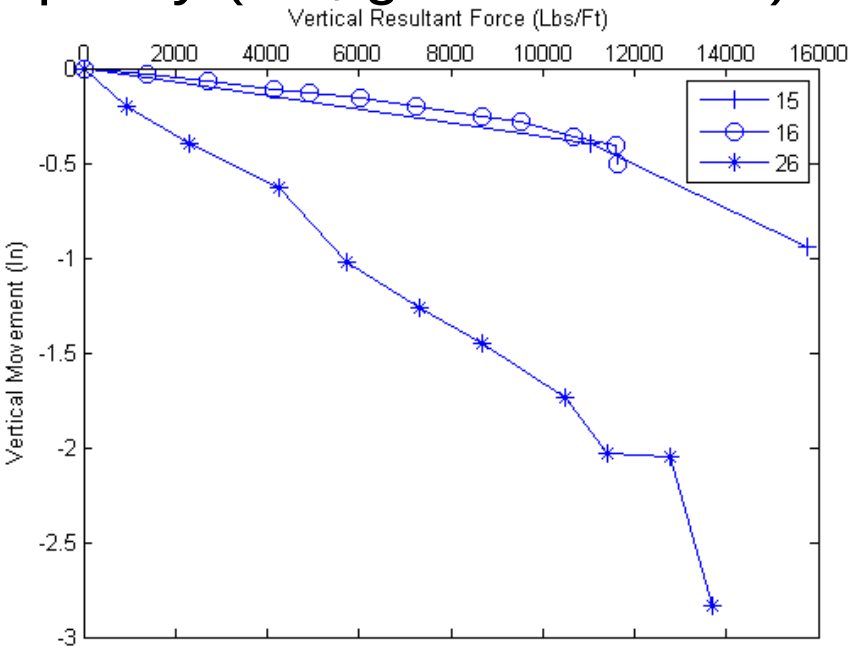
$$V = \left[\frac{W_1 + W_2 + W_3 + Q_S}{\frac{\sin(\delta)}{\tan(\theta - \phi)} + \cos(\delta)}\right] \cos(\delta)$$

Of interest: S_2, V and e

Bearing Capacity Tests

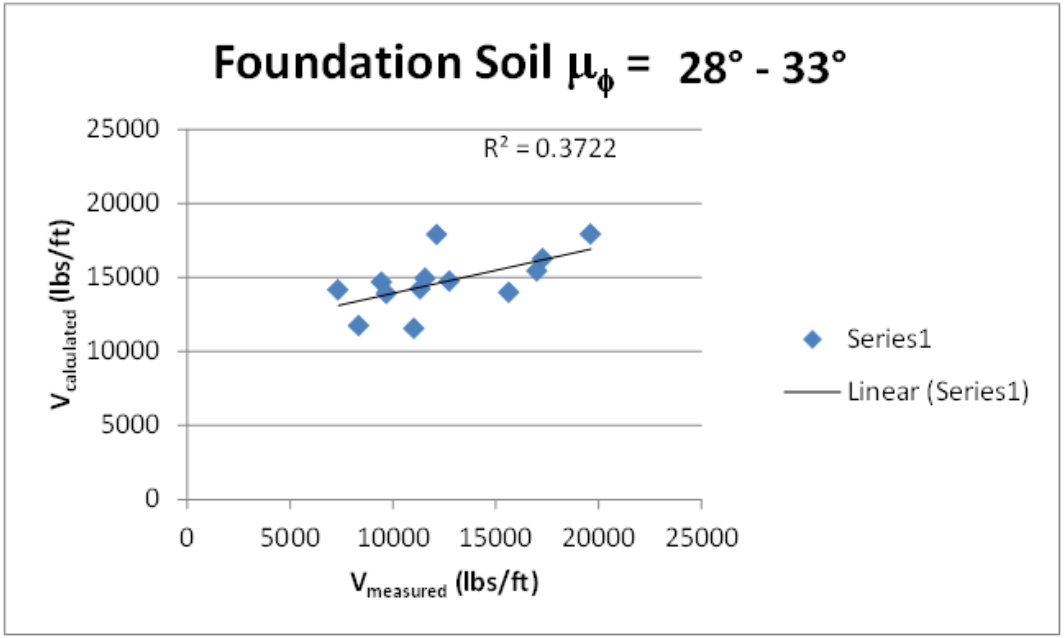
- Measured soil pressure distribution for each surcharge load
- Similar pressure distributions from full scale MSE wall test reported by Liang and Almoh'd (2004)
- The vertical resultant force, V, for each surcharge is found by taking the area under each distribution
- Load-displacement (LVDT at top of wall) plotted for bearing soil response and determine capacity (i.e., general failure)





Bearing Capacity Tests

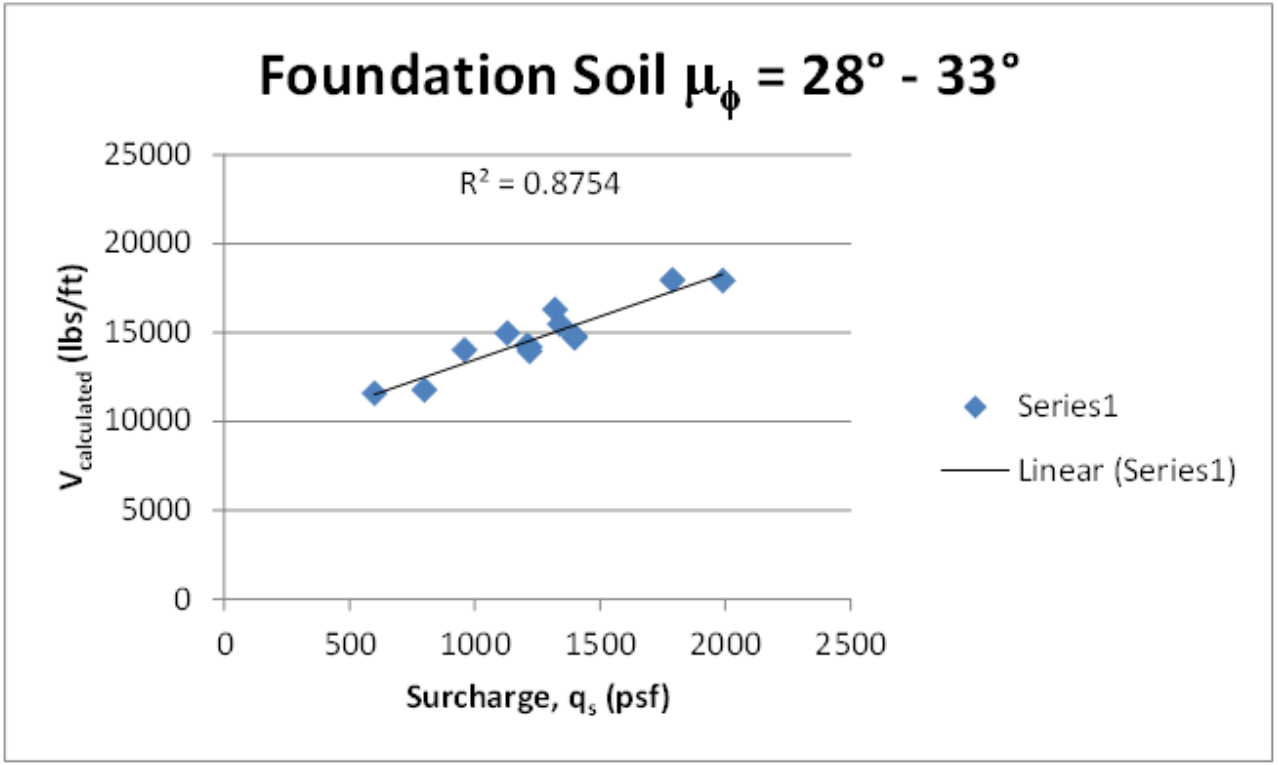
- $V_{calculated}$ versus $V_{measured}$
- $\sigma_{measured}$ = 3757 lbs/ft > $\sigma_{calculated}$ = 1910 lbs/ft
- $\sigma_{measured}$ attributed to approximation of pressure distribution and sensor error



- Backfill μ_γ and μ_ϕ range 93 pcf – 99 pcf and 28° and 33°

Bearing Capacity Tests

- $V_{calculated}$ versus increasing surcharge, q_s
- $V_{calculated}$ were used for LRFD Φ



- Backfill μ_γ and μ_ϕ range 93 pcf – 99 pcf and 28° and 33°

Methods for N_γ and Load Inclination Factor (i_γ) (Bowles, 1996)

- Bearing capacity equation: $q_u = \frac{1}{2}\gamma L' N_\gamma i_\gamma$

- Terzaghi: $N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)$

  where $K_{p\gamma}$ is earth pressure coefficient

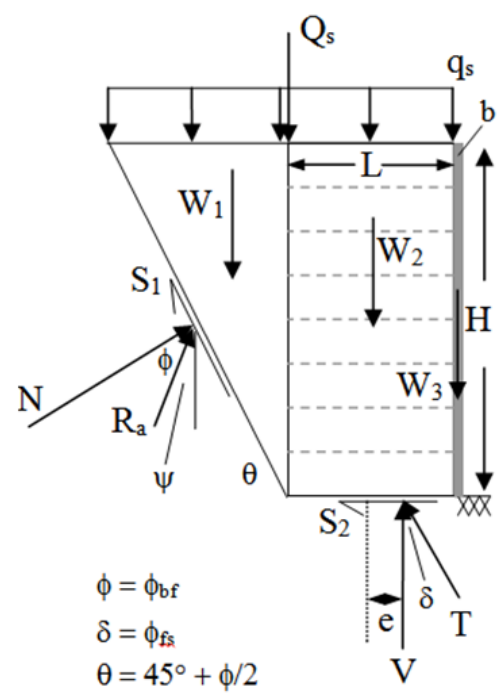
  ϕ is angle δ

  No suggested load inclination factor

- Meyerhof: $N_\gamma = (N_q - 1)\tan(1.4\phi)$

  where $N_q = e^{\pi\tan\phi}\tan^2\left(45 + \frac{\phi}{2}\right)$

$$i_\gamma = \left(1 - \frac{\theta^\circ}{\phi^\circ}\right)^2$$



where θ° is the angle of the resultant from the vertical

ϕ° is the foundation soil's friction angle

Review of N_γ Methods and Load Inclination Factors (i_γ)

- Hansen:

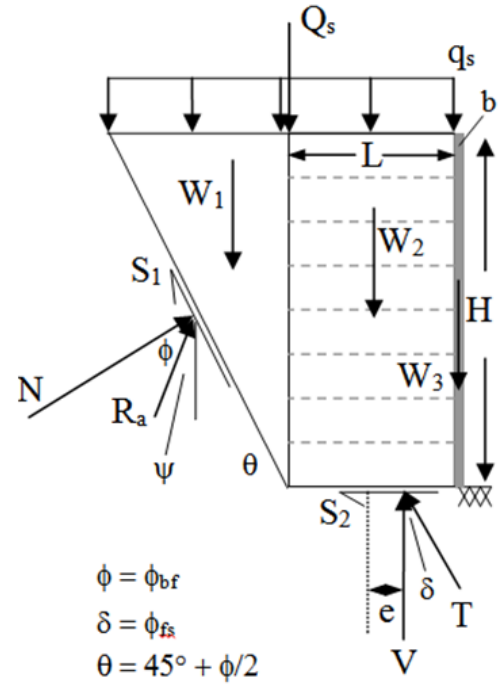
$$N_\gamma = 1.5(N_q - 1)\tan\phi$$

where N_q is same as Meyerhof

$$i_\gamma = \left(1 - \frac{0.7S_2}{V}\right)^\eta$$

$$2 \le \eta \le 5$$

where $\eta = 2$ was selected



- Vesic:

$$N_\gamma = 2(N_q + 1)\tan\phi$$

where N_q is same as Meyerhof

$$i_\gamma = \left(1 - \frac{S_2}{V}\right)^{m+1}$$

m = (2+B/L)/(1+B/L)

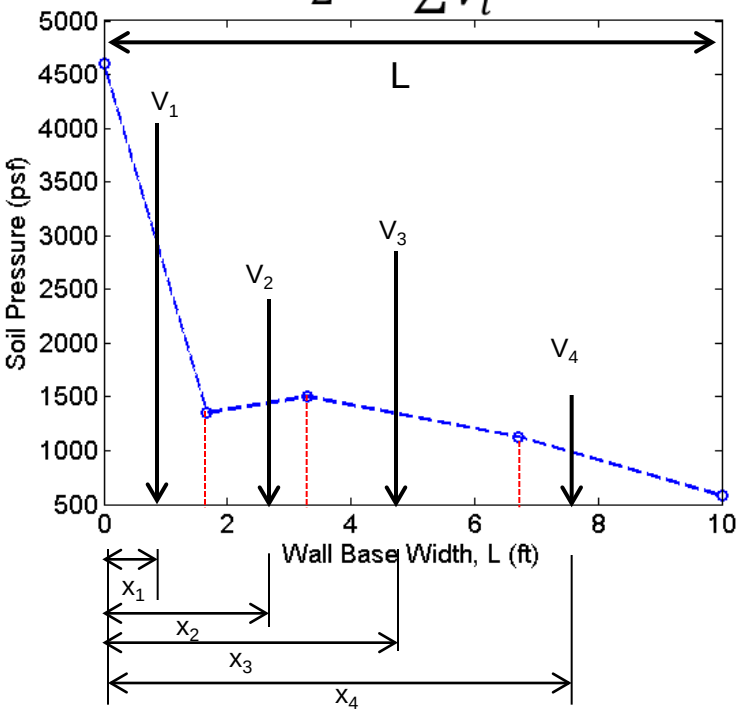
where B is the foundation width and L is the unit length

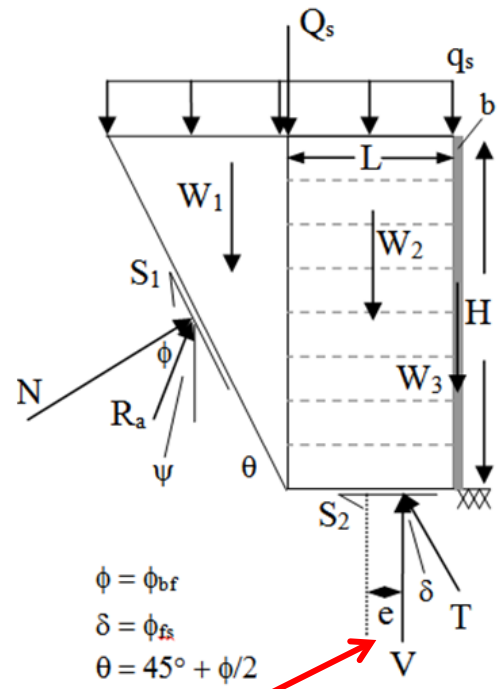
for these tests, m =1.09

Measuring and Calculating Eccentricity

- Eccentric load accounted for in bearing capacity equation with effective foundation width term, L' = L-2e
- Eccentricity of the vertical resultant load, V, can be calculated from measured soil pressure distributions through moment equilibrium about 0 ft (toe of wall)

$$e = \frac{L}{2} - \frac{\sum V_i x_i}{\sum V_i}$$





$$e = \frac{L}{2} - \frac{M_r - M_o}{V}$$

- M_r, M_o = resisting moment and overturning moment about the toe (facing elements) of wall
- V = vertical resultant force

Task 5-LRFD Φ for MSE Wall Bearing Capacity

Where

$\alpha_{EV} = 1.35$

$\alpha_{LS} = 1.75$

$\lambda_{EV} = 1.12$

$\lambda_{LS} = 1.2$

α_{EV} and α_{EV} are AASHTO (2009) recommended values

λ_{EV} calculated based tests and geostatic stress ($\sigma_o = \Sigma\gamma H$)

λ_{LS} conservative value recommended by Nowak (1999)

$\Phi = 0.65$ (AASHTO, 2009)

μ_ϕ=28°-33° (n=13)		
Terzaghi ($CV_R = 0.30$)		
p_f	λ_R	Φ
0.01	0.25	0.14
0.001	0.25	0.11
Meyerhof ($CV_R = 0.34$)		
p_f	λ_R	Φ
0.01	2.69	1.40
0.001	2.69	1.00
Hansen ($CV_R = 0.28$)		
p_f	λ_R	Φ
0.01	1.1	0.65
0.001	1.1	0.50
Vesic ($CV_R = 0.32$)		
p_f	λ_R	Φ
0.01	2.06	1.10
0.001	2.06	0.84

- Terzaghi Φ are overly conservative, low mean bias partially attributed to no load inclination reduction
- Meyerhof $\Phi \geq 1$ due to high mean bias
- Hansen Φ close to AASHTO recommended (0.65), mean bias suggests good method, accounts for load inclination taking $0.7(S_2/V)$
- Vesic Φ = 1.1 – 0.84, however mean bias > 2
- Only 13 data points for a range of foundation soil μ_ϕ (28° -33°)…need more tests

Future Work

- Centrifuge tests
  - More tests of MSE wall bearing capacity models to assess CV_R and bias in resistance, λ_R
  - Tests of MSE wall near slope bearing capacity
- Develop resistance factors, Φ
  - Analytical forms of Φ as a function of H, L, μ_ϕ, μ_γ, CV_ϕ, CV_γ
  - Develop and suggest for external MSE wall stability and cases of proximity to sloping soil embankments

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Thank You!

Questions?