Development of LRFD Resistance Factors for External Stability of Mechanically Stabilized Earth (MSE) Walls

Final: FDOT BDK75-977-22

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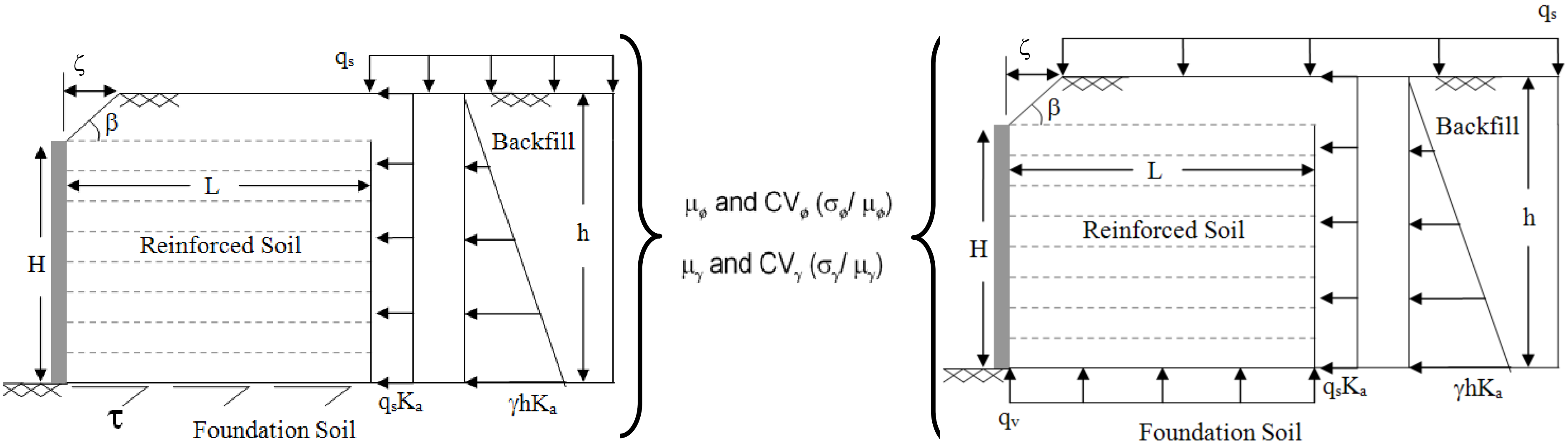
Scope of Work

- Perform centrifuge tests of MSE wall external stability which include soil variability
  - Sliding stability
  - Bearing capacity (flat ground)
  - Bearing capacity (on embankments)
- Analyze and validate test results
  - Calculate dead load factors: horizontal and vertical soil pressures
  - Quantify CV_Q, CV_R (load and resistance)
  - Quantify bias: λ_Q and λ_R for conventional methods
- Validate methods for bearing of walls on embankments
- Develop resistance factors (Φ) for stability cases tested

Background-Conventional Methods

Sliding Bearing



Use conventional analytical expressions where load and resistances are factored with AASHTO recommended values

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

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

K_a from Rankine or Coulomb

Background-AASHTO LRFD

- Governing equation

$$\Phi \ R_n \quad \geq \ \eta \ \Sigma\alpha_i Q_i$$

η = 1.0 - redundancy
α = load factor (dead, vertical, horizontal, surcharge, etc.)
Q = load or force effect
Φ = resistance factor
R_n = nominal resistance (force)

- Load factor equation

$$\alpha = \lambda(1 + nCV)$$

λ = bias in load
n = constant
CV = σ/μ (measured)

Background-AASHTO LRFD

- Resistance Factor (Φ) Equation (FHWA, 2001 and Styler, 2006)

$$\Phi = \frac{\lambda_R(\alpha_D q_D + \alpha_L q_L)\sqrt{\frac{(1+CV_q{}^2)}{(1+CV_R{}^2}}}{(\lambda_D q_D + \lambda_L q_L)\exp^{\beta\ln((1+CV_R{}^2)(1+CV_q{}^2))}}$$

Derived with First Order Second Moment (FOSM) and for lognormal load and resistance

$$CV_q = \frac{(q_D\lambda_D CV_D)^2 + (q_L\lambda_L CV_L)^2}{(q_D\lambda_D)^2 + 2q_D q_L\lambda_D\lambda_L + (q_L\lambda_L)^2}$$

$$CV_R = \frac{\sigma_R}{\mu_R}$$

α = Dead and live load factors

λ = bias (measured/predicted)

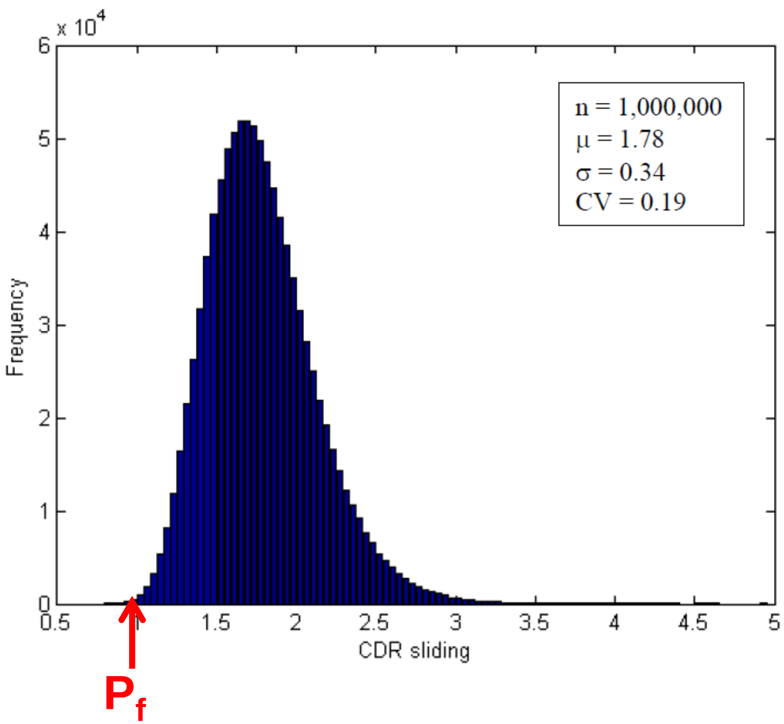
β = Reliability

Soil Property Parametric Study

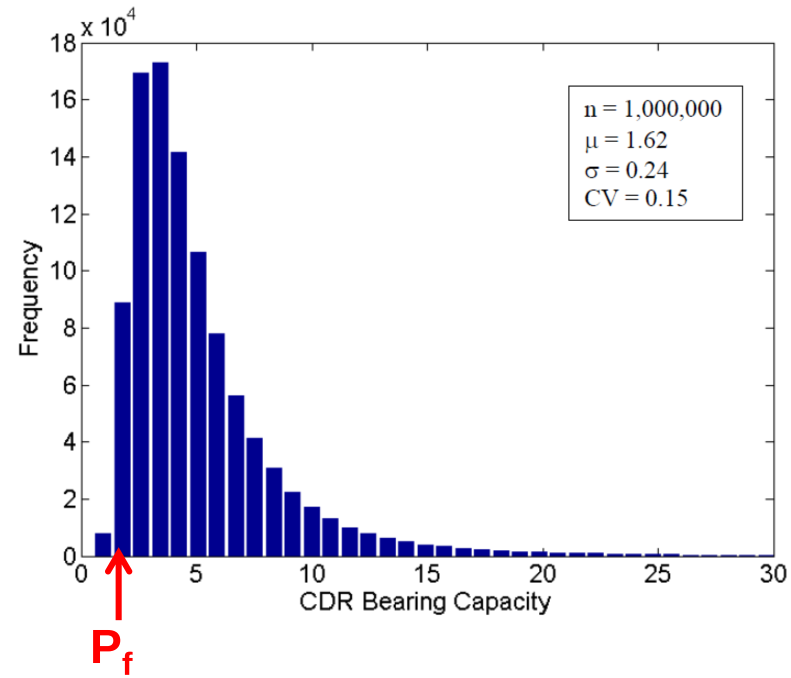
	Baseline Parameters		Range of Parameters		
	Mean (μ)	Coefficient of Variation (CV)	Mean (μ)	Coefficient of Variation (CV)	Distribution
Retained Soil ϕ	30°	10%	20° - 40°	5% - 20%	Lognormal
Retained Soil γ	105 pcf	5%	95 pcf - 120 pcf	5% - 20%	Lognormal
Foundation Soil ϕ	35°	10%	20° - 40°	5% - 20%	Lognormal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

Stability Simulations-Monte Carlo

Sliding



Bearing



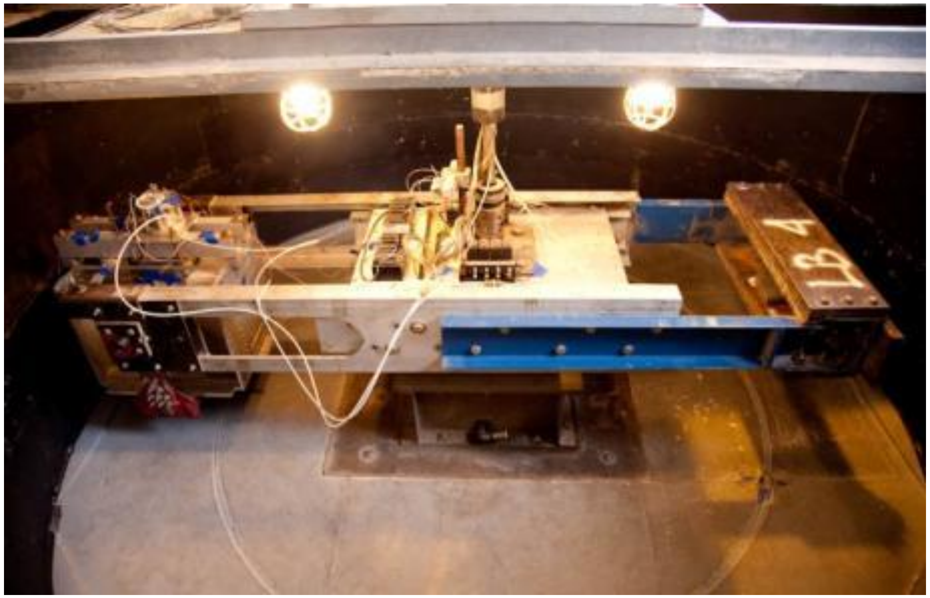
Greatest influence on P_f from:

- CV_ϕ and μ_ϕ of backfill and foundation soil;
- CV_γ backfill

Greatest influence on P_f from:

- CV_ϕ and μ_ϕ of foundation soil
- μ_ϕ of retained soil

Centrifuge Tests

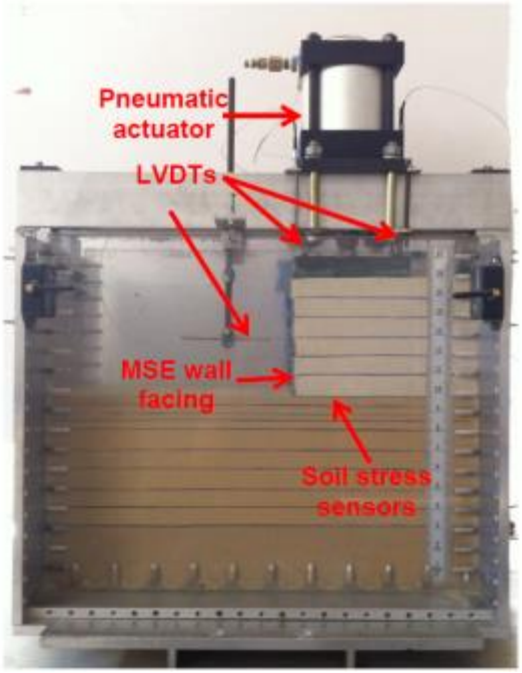


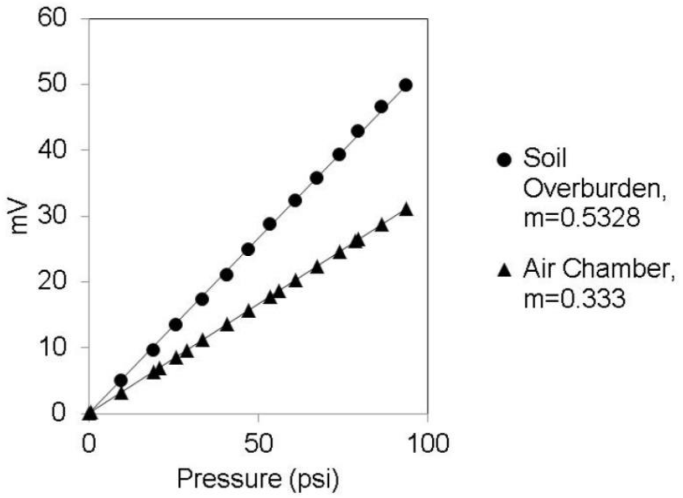
Scaling laws

Property	Prototype	Model
Acceleration (L/T^2)	1	N
Linear Dimensions (L)	1	1/N
Area (L^2)	1	$1/N^2$
Volume (L^3)	1	$1/N^3$
Mass (M)	1	$1/N^3$
Force (ML/T^2)	1	$1/N^2$
Unit Weight (M/L^2T^2)	1	N
Density (M/L^3)	1	1
Stress (M/LT^2)	1	1
Strain (L/L)	1	1
Moment (ML^2/T^2)	1	$1/N^3$

- 2.6 m diameter; 12.5 G-Ton capacity beam centrifuge
- Model heights up to 24 in, widths up to 20 in
- Hydraulic system for double acting pneumatic pistons
- 12-Channel wireless data acquisition

Instrumentation





Factor	Required ratios	Measured Ratios
Aspect ratio	T/D < 1/5 (Experimentation Station, 1944) < 1/10 (Dunnicliff, 1988)	1/7.5 – 1.5/7.5 < 1/5
Active diameter	$d/D_{50} > 10$	6/0.2 > 10
Sensor-soil stiffness ratio	> 0.5	28.5×10^3 ksi/(0.6 – 4 ksi) > 0.5
Active diameter/Deflection	$d/\Delta > 2000 - 5000$	6/0.002 > 2000

- Stress sensor requires calibration for use in soil
- Performed in centrifuge utilizing increased G environments
- Embedded sensor will influence measurements
- Factors must be satisfied for reliable output

Sliding Stability Models

s_v = 1.5 inch

s_h = 2 inch

$\#rows$ = 4

w_r = 0.25 inch

t_r = 0.0125 inch

f'_y = 35,000 psi (reinforcement)

f'_y = 2,324 psi (connection)

H = 6 inches

L = 6 inches

$\Phi_{pullout}$ = 0.90

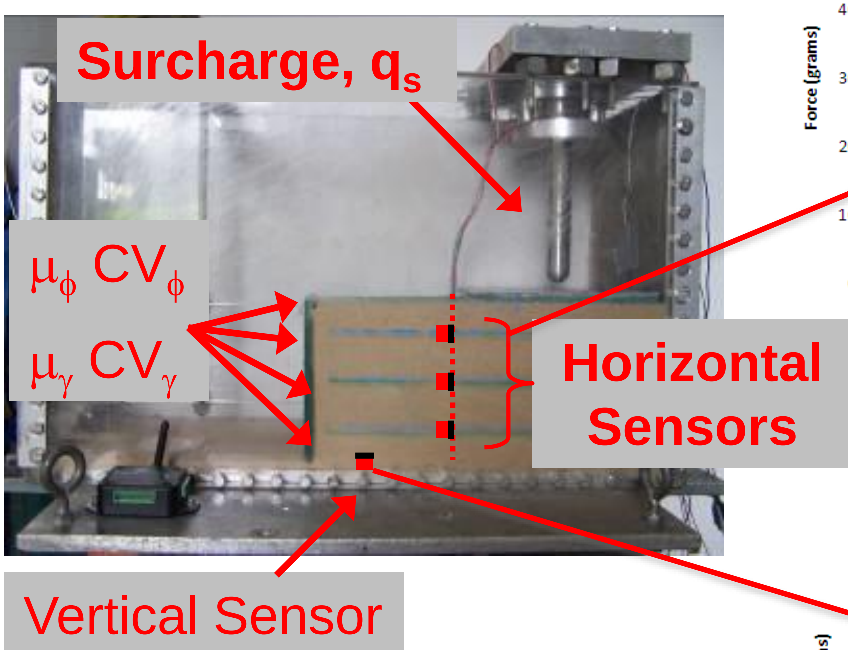
$\Phi_{rupture}$ = 0.75

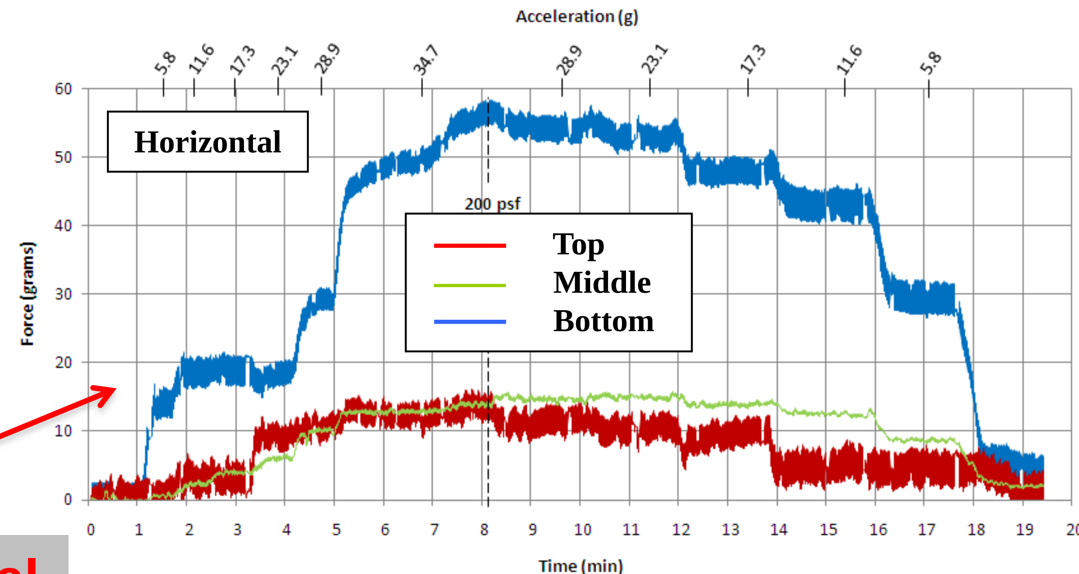
γ_{DL} = 1.35

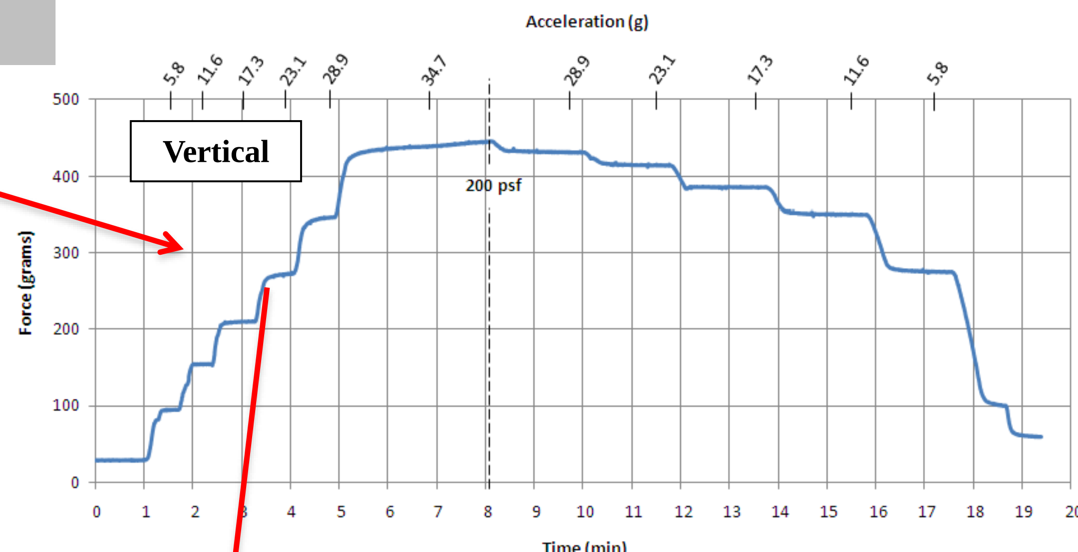
Designed for stability against pullout and rupture failure

$CDR_{pullout,\ rupture}$ (capacity/demand) > 2

Sliding Stability Tests





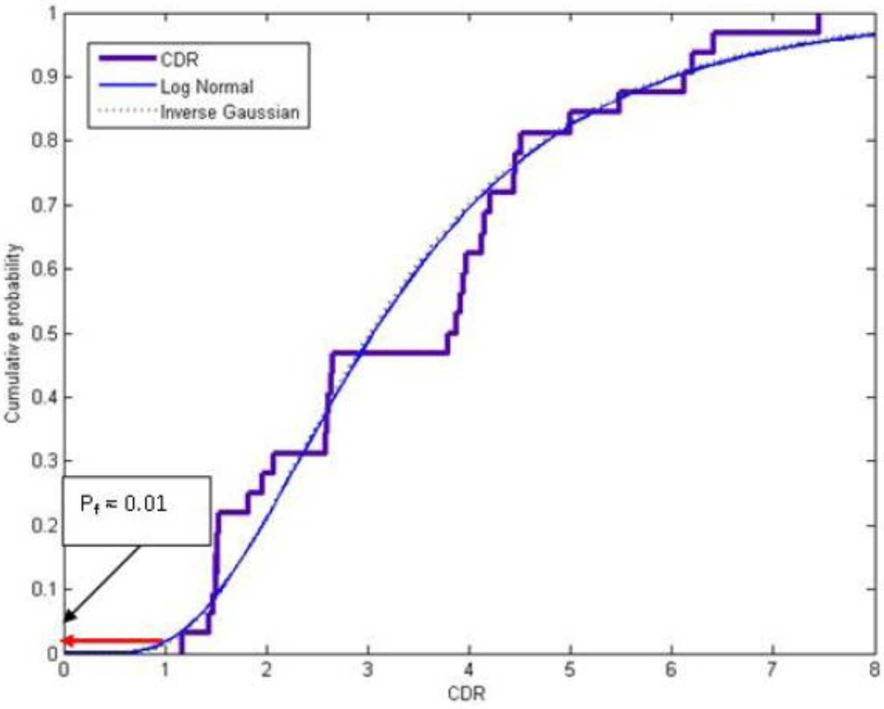


Acceleration (g)	Prototype Wall Height (ft)
34.67	17.57
28.90	14.64
23.11	11.71
17.34	8.79
11.56	5.86
5.78	2.93

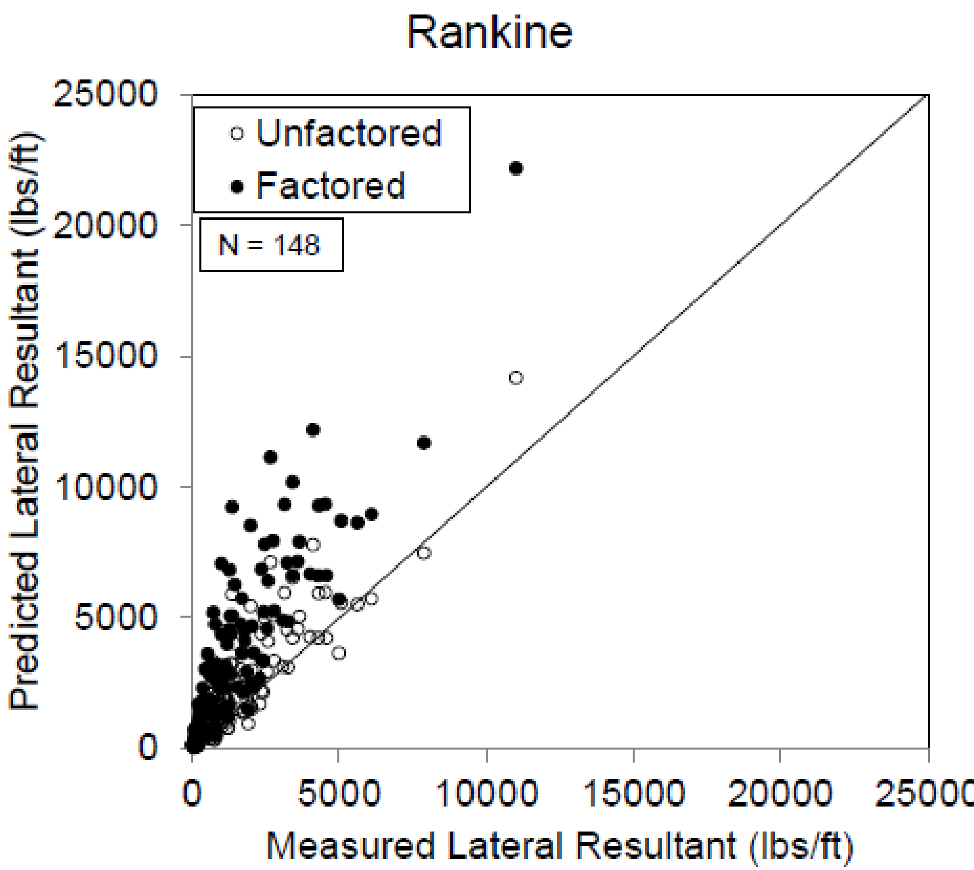
$$\tau = \sigma_N \tan(\phi)$$

Sliding Stability Test Results

- $CDR_{measured}$ grouped by $\mu_{\phi bf}$ (backfill)
- $\mu_{\phi bf} = 32°$ and $CV_{\phi} = 11\%$
- K-S (Kolmogorov-Smirnov) fit test $\alpha = 5\%$ showed both Lognormal and Inverse Gauss to fit



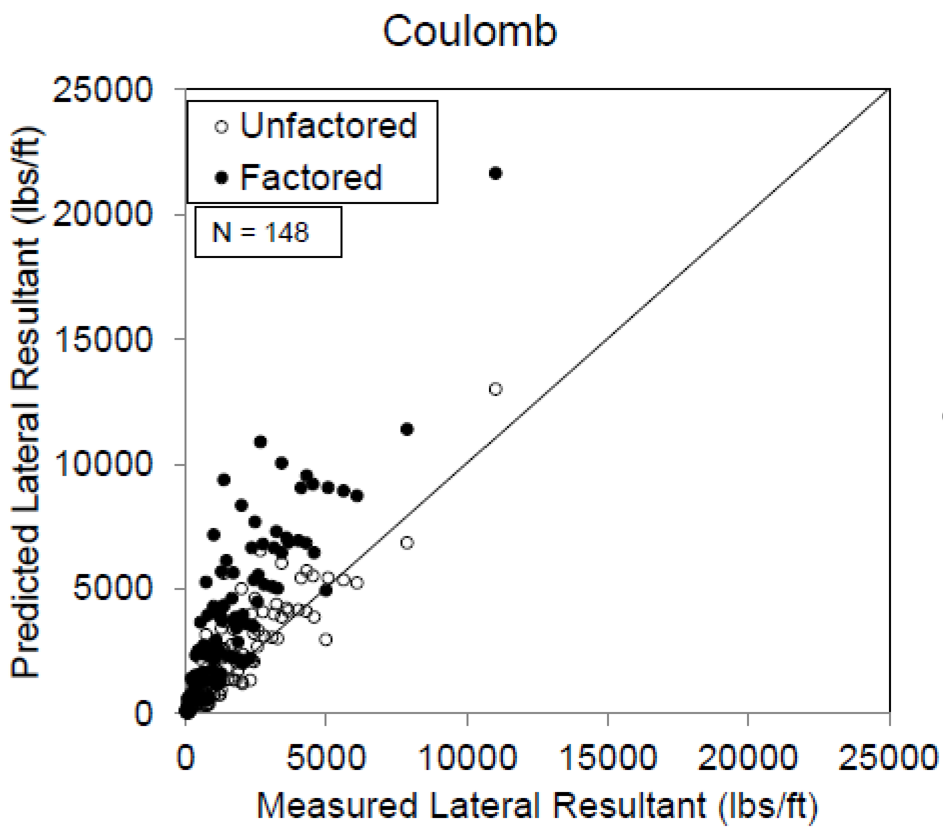
Horizontal Dead Load Factor, γ_{DL}



$$\sigma'_{H_{Rankine}} = \sigma'_V K_a$$

- For $\lambda = 0.70$, CV = 0.62 $\gamma_{EH} = 1.52$
- AASHTO (2012) recommends $\gamma_{EH} = 1.5$

Horizontal Dead Load Factor, γ_{DL}



$$\sigma'_{H_{Coulomb}} = \sigma'_V K_a$$

- For λ = 0.78, CV = 0.56 γ_{EH} = 1.63
- AASHTO (2012) recommends γ_{EH} = 1.5

LRFD Φ for Sliding Stability

Rankine

Table 3-4 Calculated Φ values based on Rankine's loading (backfill: $\mu_\phi = 32°$ and $CV_\phi = 11.7\%$) L/H =1

Wall Height (ft)	No. Values (n)	Measured Resistance		Measured Load		Bias = (Measured/Predicted)			Load Factors			Φ
		μ_R (lb/ft)	CV_R	μ_Q (lb/ft)	CV_Q	λ_R	λ_D	λ_L	γ_{EV}	γ_{EH}	γ_{LS}	
8	9	2891.90	0.81	1908.10	0.60	1.21	1.4	1.2	1	1.5	1.75	0.79
11	9	4338.90	0.80	2799.63	0.81	1.04	1.17	1.2	1	1.5	1.75	0.94
14	6	4671.10	0.76	2504.57	0.16	0.6	0.7	1.2	1	1.5	1.75	0.74

Coulomb

Table 3-5 Calculated Φ values based on Coulomb's loading (backfill: $\mu_\phi = 32°$ and $CV_\phi = 11.7\%$) L/H =1

Wall Height (ft)	No. Values (n)	Measured Resistance		Measured Load		Bias = (Measured/Predicted)			Load Factors			Φ
		μ_R (lb/ft)	CV_R	μ_Q (lb/ft)	CV_Q	λ_R	λ_D	λ_L	γ_{EV}	γ_{EH}	γ_{LS}	
8	9	2891.90	0.81	1908.10	0.60	1.21	1.8	1.2	1	1.6	1.75	0.63
11	9	4338.90	0.80	2799.63	0.81	1.04	1.46	1.2	1	1.6	1.75	0.64
14	6	4671.10	0.76	2504.57	0.16	0.6	0.8	1.2	1	1.6	1.75	0.68

<u>AASHTO recommended value</u>

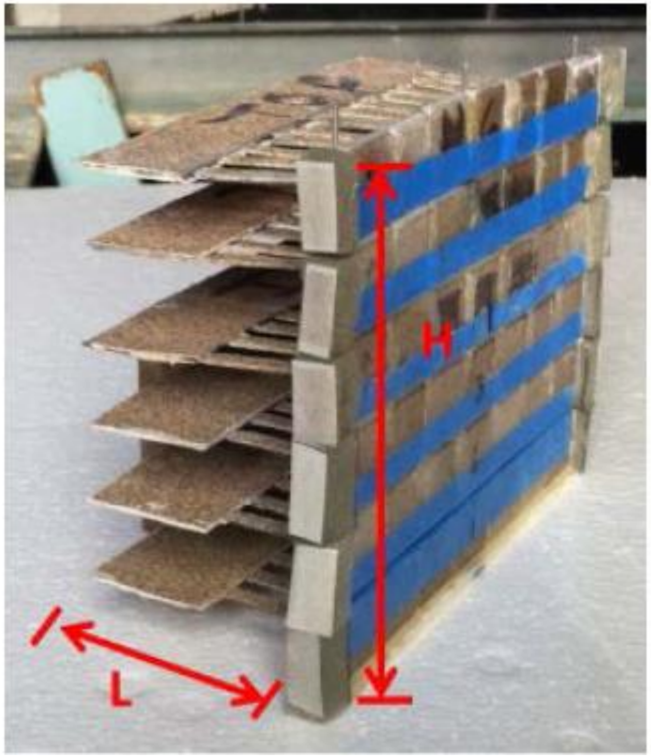
Φ = 1.0

<u>UF values</u>

Rankine: Φ = 0.74 – 0.94

Coulomb: Φ = 0.63 – 0.68

Bearing Stability Models



s_v = 0.78 inch

s_h = 0.47 inch

$\#rows$ = 6

w_r = 0.25 inch

t_r = 0.0125 inch

f'_y = 35,000 psi (reinforcement)

f'_y = 2,324 psi (connection)

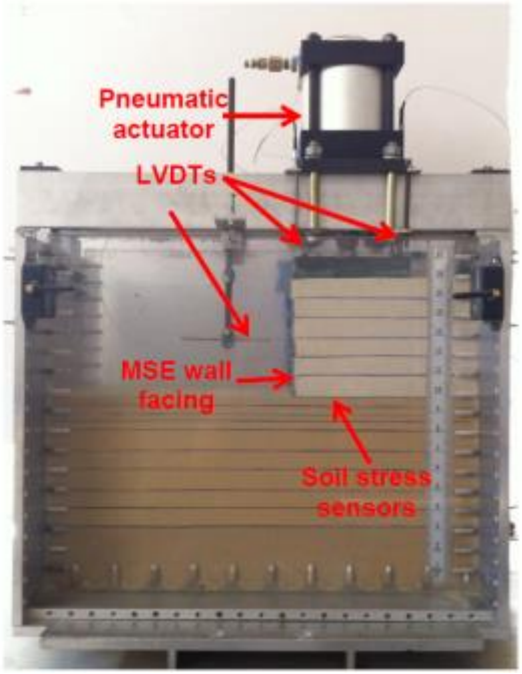
H = 6 inches

L = 3 inches

$\Phi_{pullout}$ = 0.90

$\Phi_{rupture}$ = 0.75

γ_{DL} = 1.35

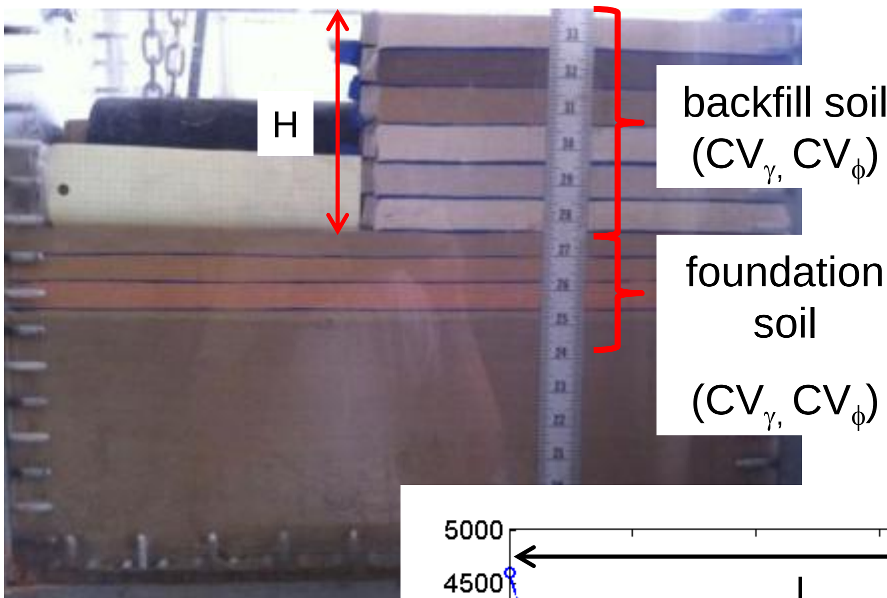


Designed for stability against pullout and rupture failure

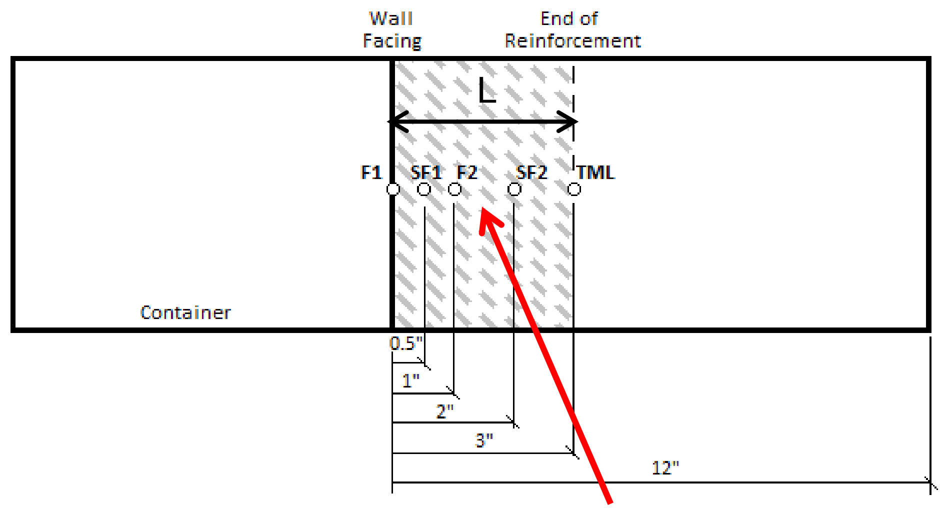
$CDR_{pullout,\ rupture}$ (capacity/demand) > 2

Bearing Stability Tests

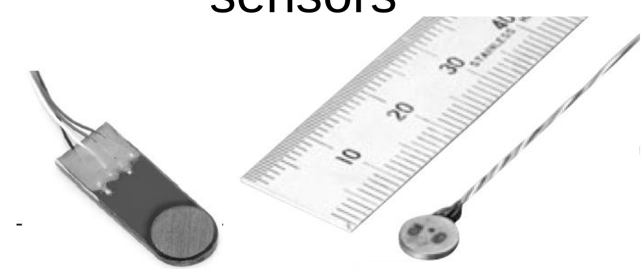
Side View



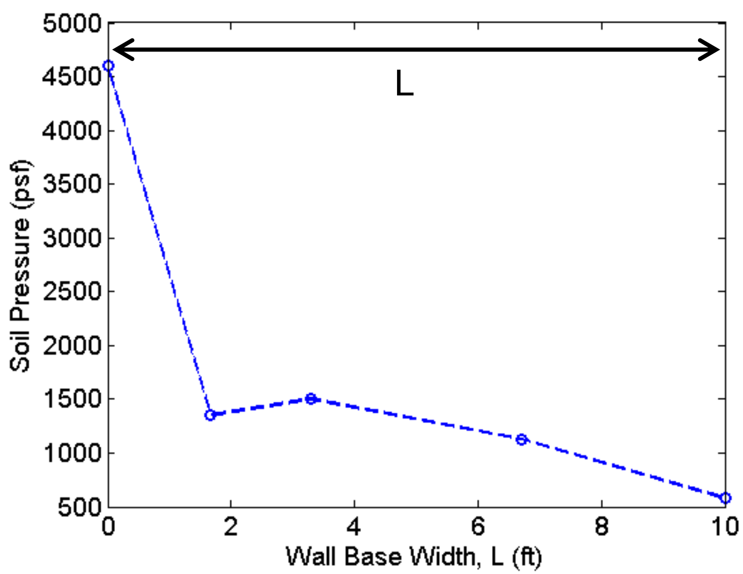
Plan View



Miniature soil pressure sensors

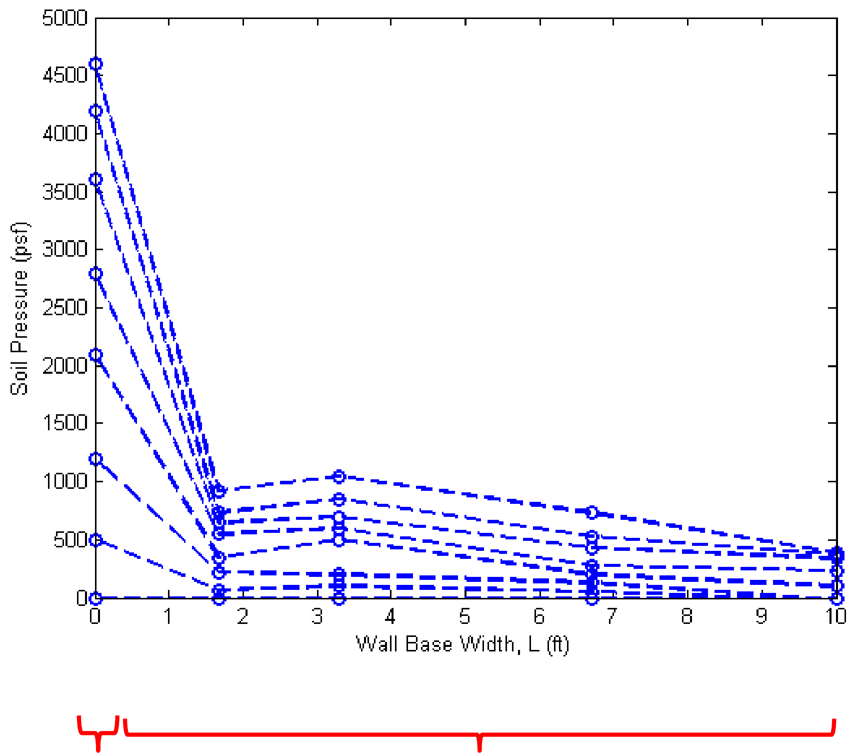


Measured soil pressure distribution

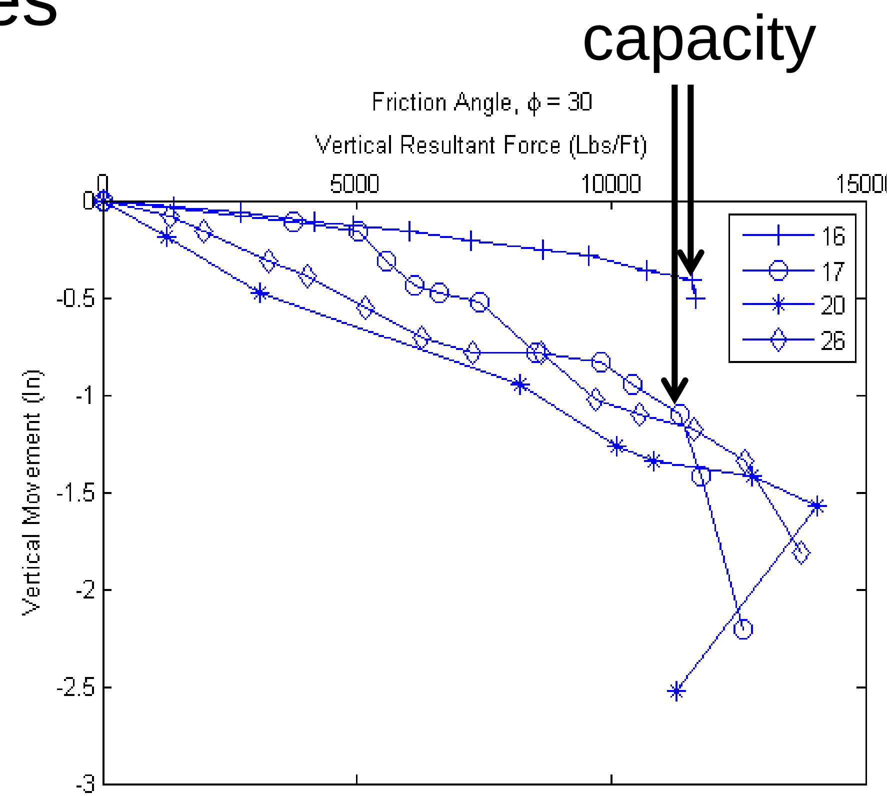


Bearing Stability Test Results

Load-Displacement curves



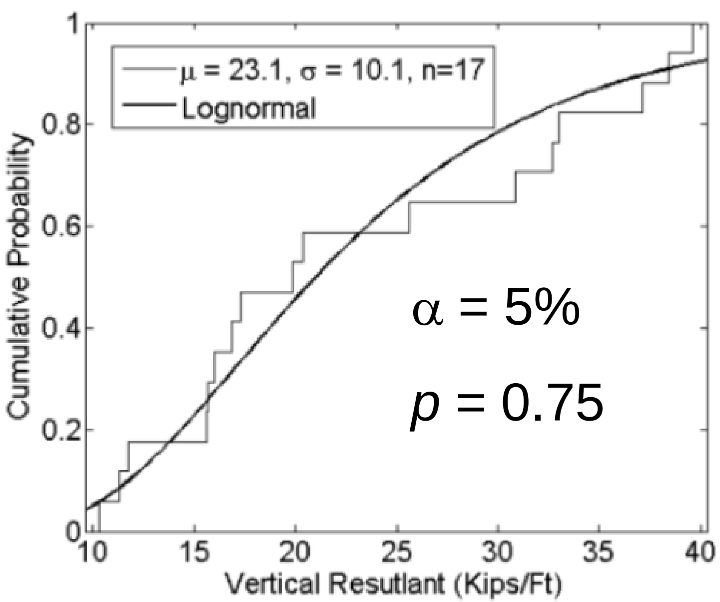
Rigid wall facing   Flexible reinforced soil

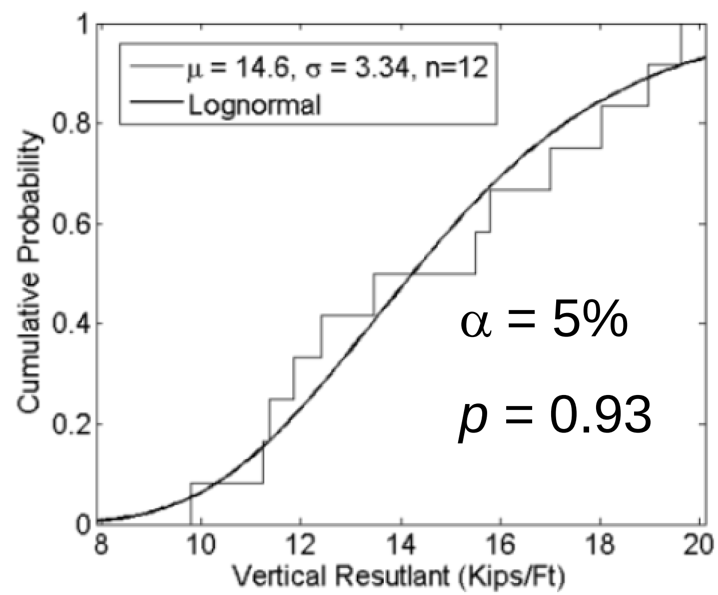


$$F_V = \int \sigma_V \, dL = \sum \sigma_V \Delta L$$

Bearing Stability Test Results

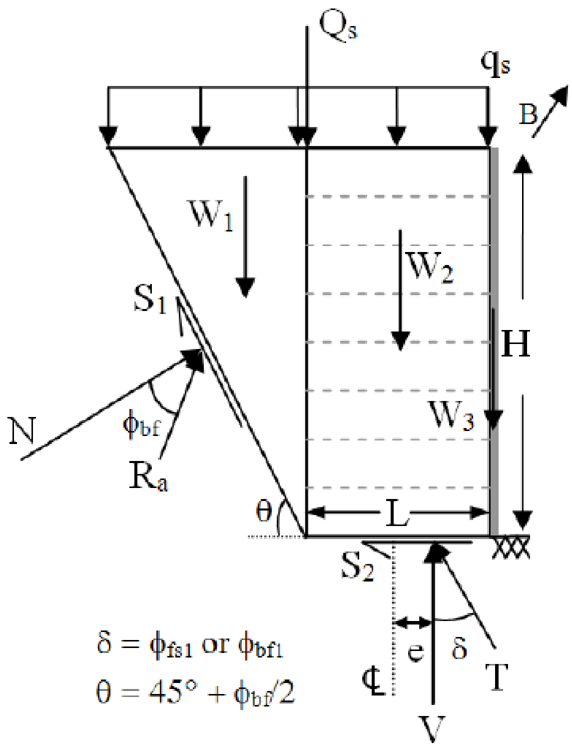
- $V_{measured}$ (capacity) grouped by $\mu_{\phi fs}$ (foundation soil)
- $\mu_{\phi fs}$ = 26° -30° and $\mu_{\phi fs}$ = 31° -33°
- K-S (Kolmogorov-Smirnov) fit test α = 5%

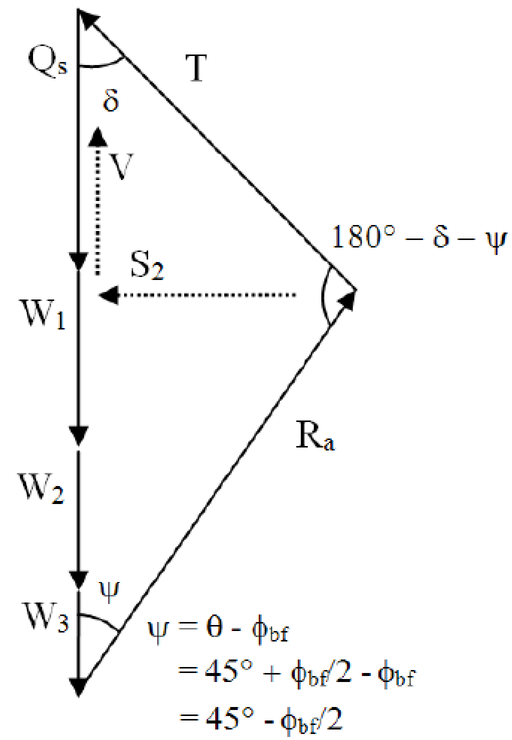




Validation-Force Equilibrium

- Failure of wall can be described by a planar rupture surface through backfill (observed in tests)
- $V_{calculated}$ from force polygon and measured weights



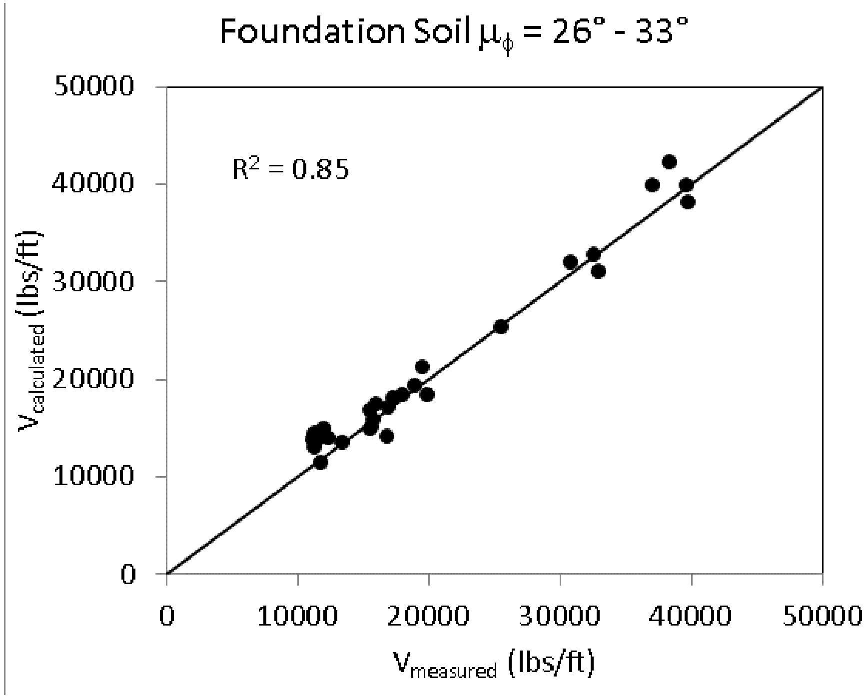


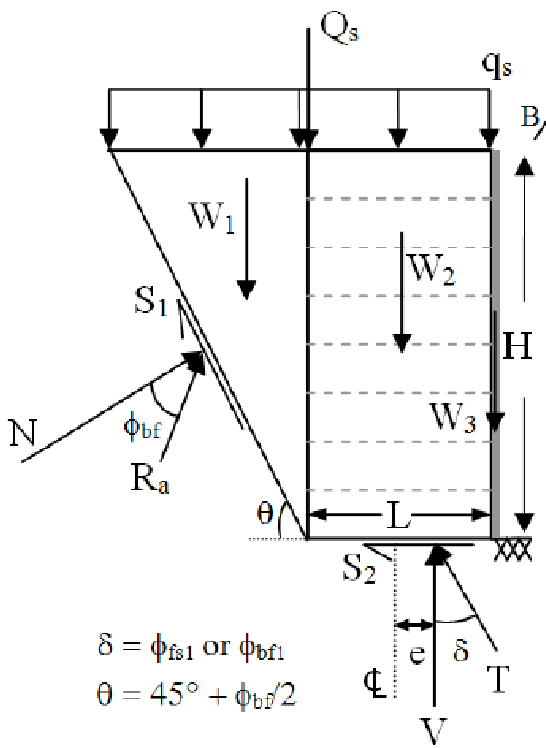
Of interest: S_2, V and e

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

Validation-Force Equilibrium

- Good correlation between $V_{measured}$ and $V_{calculated}$
- Model is accurate representation
- Useful for investigating S_2, eccentricity (e) and angle of inclination (δ)

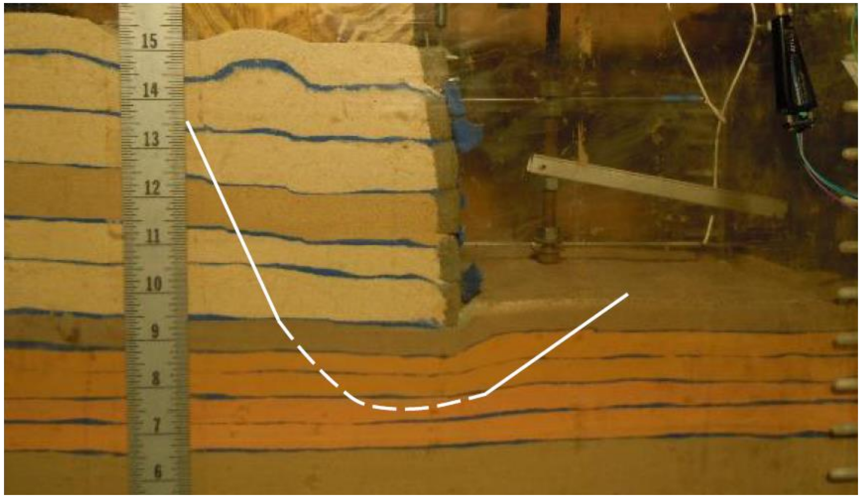




- Backfill μ_γ and μ_ϕ range 93 pcf – 99 pcf and 28° and 33°

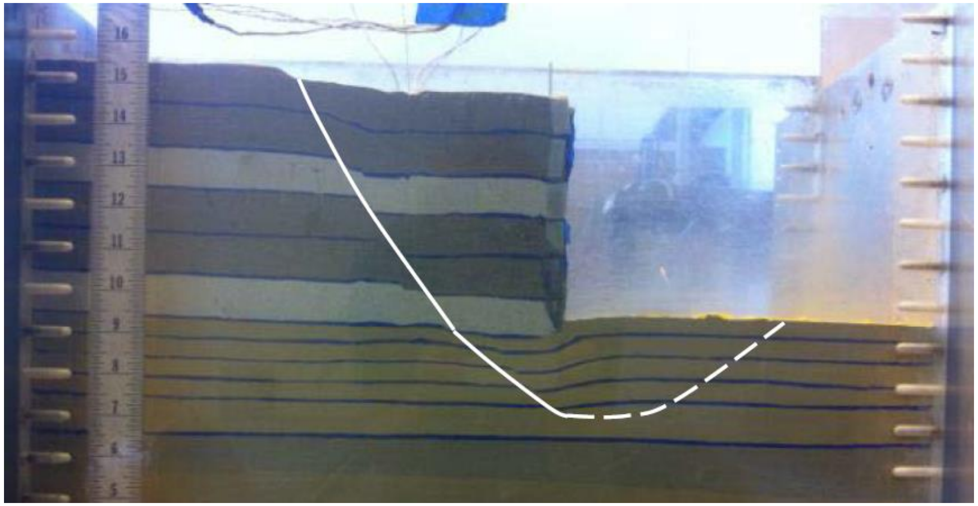
Effects of Load Inclination

- Inclined loads (T in MSE wall and wedge) reduce length of bearing rupture surface i.e., reduced capacity
- Sokolovski (1960) showed analytically for $\delta = 0° - 20°$ depth of rupture 0.78L to 0.3L and lateral extents 1.9L to 0.6L, respectively



Rupture surface in test 15 ($\delta = 30°$ and $\mu_\phi = 28°$): Dashed line is the estimated surface, Solid line is offset from observed surface

- Depth of rupture ≈ 0.5L
- Lateral extent ≈ 0.67L



Rupture surface in test 42 ($\delta = 25°$ and $\mu_\phi = 28°$): Dashed line is the estimated surface, Solid line is offset from observed surface

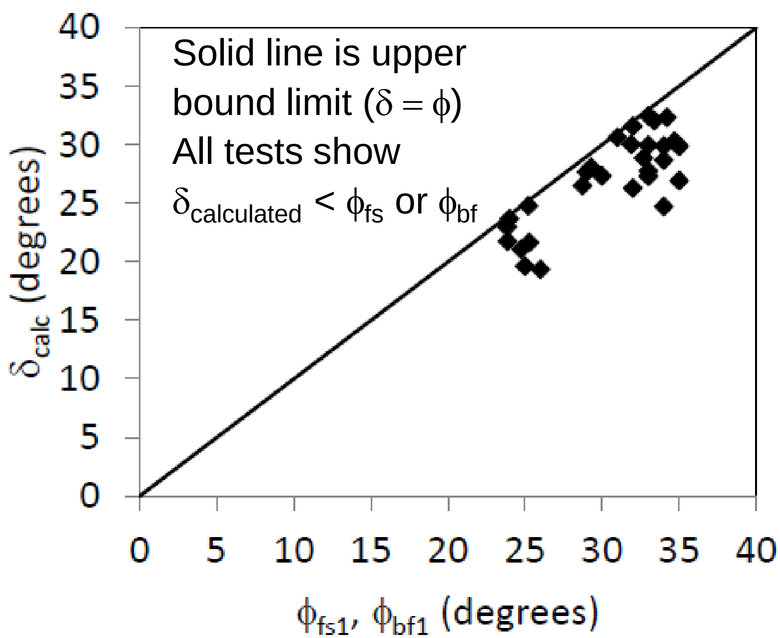
- Depth of rupture ≈ 0.7L
- Lateral extent > 0.67L

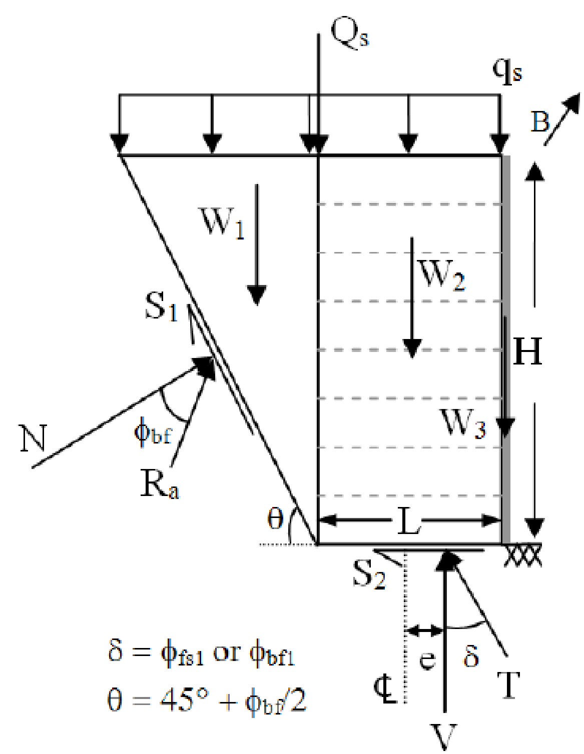
Angle of Load Inclination, δ

- Horizontal force equilibrium based on force polygon gives S_2

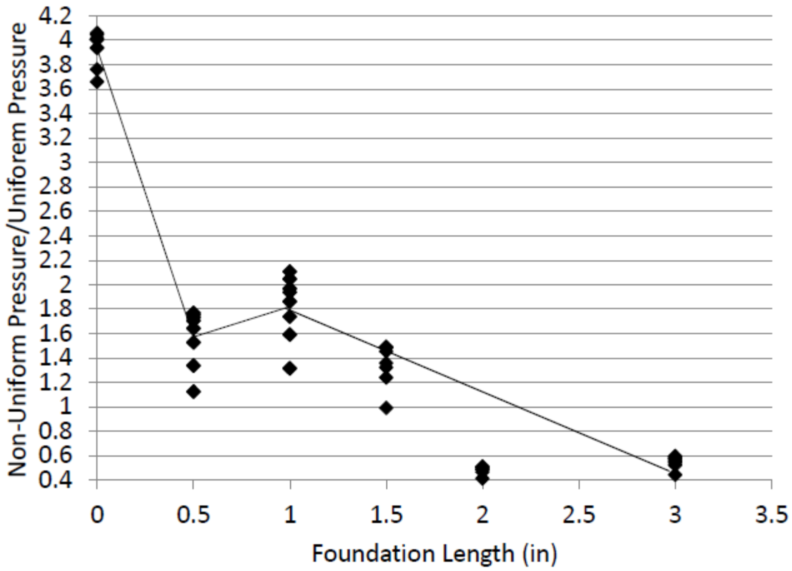
$$S_2 = (W_1 + W_2 + W_3 + Q_S - V_{meas})\tan(\theta - \phi)$$

- Using $V_{measured}$ and $\tan^{-1}(S_2/V_{measured})$, δ is back calculated for comparison to the smaller of ϕ_{fs} and ϕ_{bf} at interface between foundation soil and backfill





Observed and Predicted Vertical Dead Load Stresses Beneath MSE Wall & Dead Load Factor, γ_{DL}



$$\frac{\sigma'_{nu}}{\sigma'_V} = 0.49(x) + 1.3 \quad \text{for } 0.5 < x \leq 1 \text{ inch}$$

$$\frac{\sigma'_{nu}}{\sigma'_V} = -0.61(x) + 2.4 \quad \text{for } 1 < x \leq 3 \text{ inch}$$

$$\sigma'_V = \gamma'_s z$$

γ_s' = soil's effective unit weight

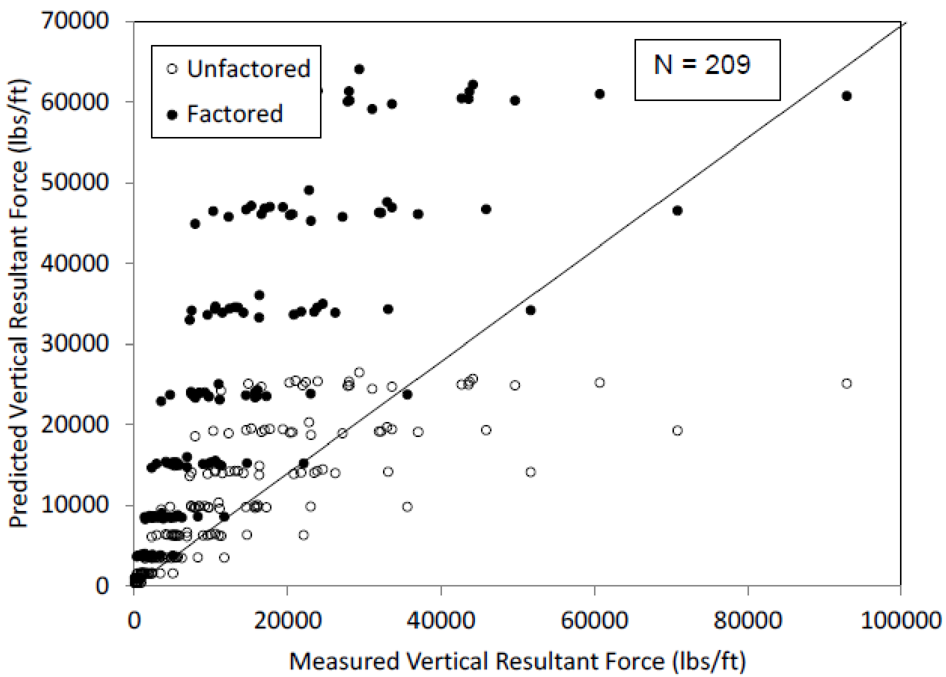
z = depth of overburden (H)

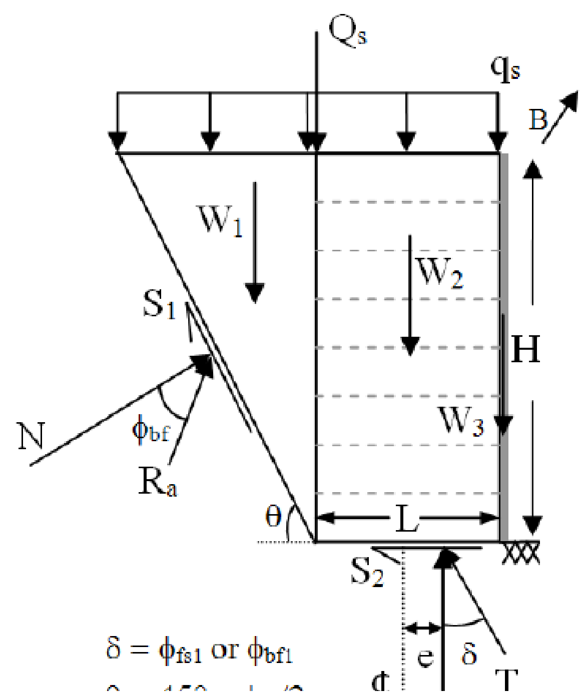
Load factor calculated with bias (λ) and CV of load (Nowak, 1995) and n = 2 (AASHTO, 2009)

$$\gamma = \lambda(1 + nCV)$$

Vertical Dead Load Factor, γ_{DL}

- Factoring the predicted load (applied vertical resultant) with γ_{EV} = 1.80 brings almost all points above 1:1 line





- For λ = 0.96, CV = 0.42 γ_{EV} = 1.80
- Bathurst et. al. proposed γ_{EV} = 1.75 from 34 tests on full scale MSE walls

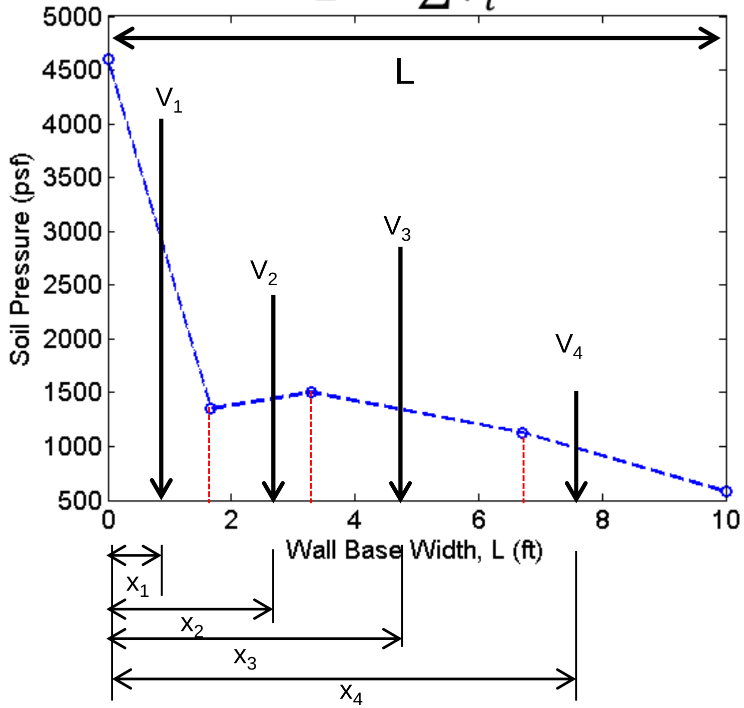
Eccentricity, e, and Effective Length, L'

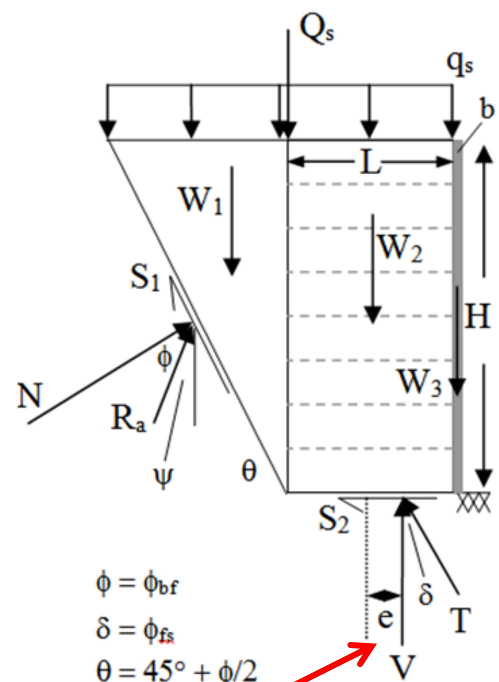
- Capacity equation: $q_u = ½\gamma L' N_\gamma i_\gamma$

  where L' = L-2e

Calculated from measured soil pressure distributions

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





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

Calculated from estimated moments and resultant vertical force

R^2 = 0.62 between measured and predicted

Bearing Capacity Analysis:

- Bearing capacity equation

$$q_u = \frac{1}{2}\gamma L' N_\gamma i_\gamma$$

- Evaluation with 7 soil self weight factors, N_γ :

- Meyerhof's

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

- Hansen's

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

- Vesic's

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

- Salgado's

$$N_\gamma = (N_q + 1)tan(1.32\phi)$$

- Eurocode 7 (2005)

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

- Michalowski (1997)

$$N_\gamma = e^{(0.66+1\tan(\phi))}tan(\phi)$$

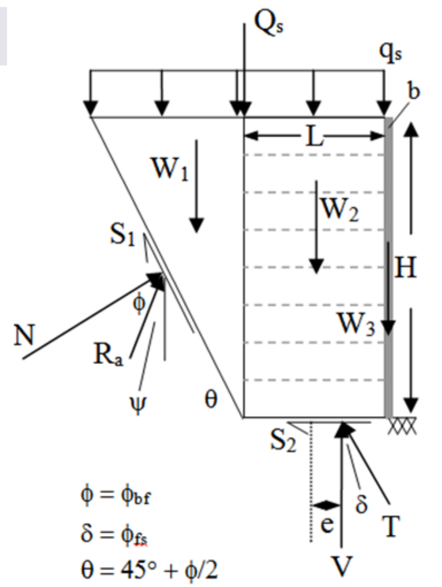
- Bolton et. al.(1993)

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

where

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

Bearing Capacity Analysis:



- Evaluation of 4 methods of load inclination factor, i_γ:

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

  $2 \leq \eta \leq 5$ (Bowles, 1996)

  $\eta = 2$ herein

- Muhn's $i_\gamma = (1 - \tan(\delta))^\eta$

  $\eta = 1$

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

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

  where

  L = foundation width

  B = unit length

  for these tests, m =1.09

- New $\frac{V_{meas}}{V_{qu_{pred}}} = \left(1 - \frac{S_2}{V}\right)^\eta$

  $i_\gamma = \left(1 - \frac{S_2}{V}\right)^{1.08}$, $26 < \phi_{found} < 30$

  $i_\gamma = \left(1 - \frac{S_2}{V}\right)^{1.55}$, $31 < \phi_{found} < 33$

LRFD Φ for Bearing Stability

$$\Phi = \frac{\lambda_R \cdot \sqrt{\frac{(1+CV_Q^2)}{(1+CV_R^2)}}(\gamma_D \cdot q_D + \gamma_L \cdot q_L)}{(\lambda_D \cdot q_D + \lambda_L \cdot q_L) \cdot e^{\beta_T \sqrt{\ln[(1+CV_R^2)(1+CV_Q^2)]}}}$$

$$CV_Q^2 = \frac{q_D^2 \cdot E[\lambda_D]^2 \cdot CV_D^2 + q_L^2 \cdot E[\lambda_L]^2 \cdot CV_L^2}{q_L^2\left(\frac{q_D^2}{q_L^2} E[\lambda_D]^2 + 2\frac{q_D}{q_L} E[\lambda_D] \cdot E[\lambda_L] + E[\lambda_L]^2\right)}$$

Summary statistics and factors used in estimating Φ

CV_D	CV_L	q_D (lbs/ft)	q_L (lbs/ft)	γ_D	γ_L	λ_D	λ_L
0.42	0.42	32,314	1,144	1.80	1.75	0.96	1.2

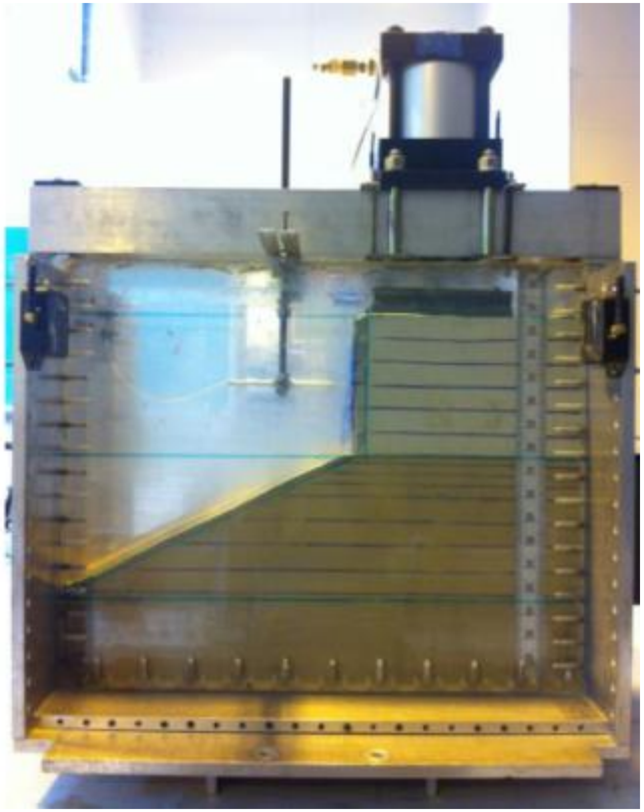
LRFD Φ's for $\mu_{\phi fs}$ = 26° - 30° using the new i_γ

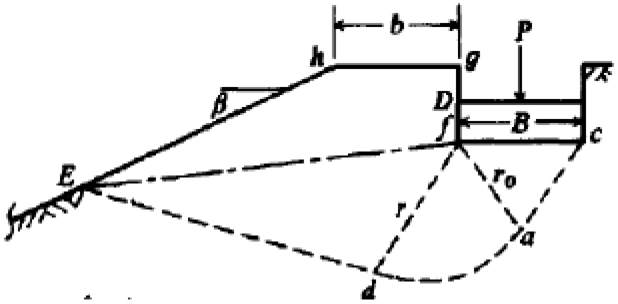
		Meyerhof	Hansen	Vesic	Salgado	Euro7	Michalowski	Bolton
CV_R		0.450	0.444	0.433	0.437	0.444	0.441	0.452
λ_R		1.93	1.98	1.29	1.82	1.48	1.39	1.75
P_f = 1%	Φ	0.986	1.020	0.682	0.956	0.765	0.721	0.889
	Φ/λ_R	0.510	0.516	0.528	0.524	0.516	0.520	0.508
P_f = 0.1%	Φ	0.668	0.694	0.467	0.652	0.520	0.491	0.602
	Φ/λ_R	0.346	0.351	0.361	0.358	0.351	0.354	0.344

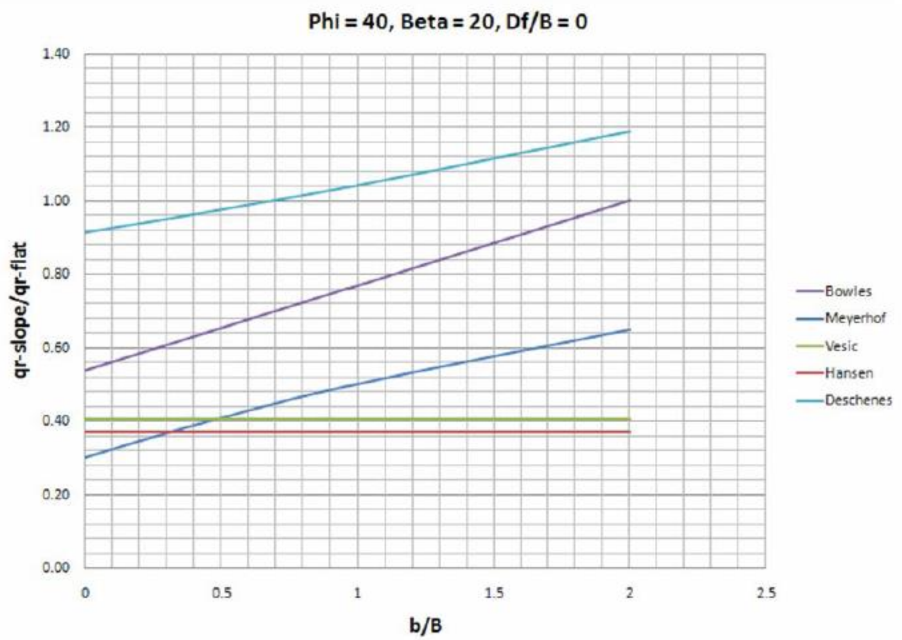
LRFD Φ's for $\mu_{\phi fs}$ = 31° - 33° using the new i_γ

		Meyerhof	Hansen	Vesic	Salgado	Euro7	Michalowski	Bolton
CV_R		0.440	0.436	0.431	0.434	0.436	0.436	0.442
λ_R		1.71	1.80	1.23	1.70	1.35	1.27	1.53
P_f = 1%	Φ	0.890	0.945	0.652	0.896	0.709	0.667	0.793
	Φ/λ_R	0.521	0.525	0.530	0.527	0.525	0.525	0.519
P_f = 0.1%	Φ	0.607	0.645	0.447	0.613	0.484	0.455	0.540
	Φ/λ_R	0.355	0.359	0.363	0.360	0.359	0.359	0.353

Tests of MSE Walls on Embankments





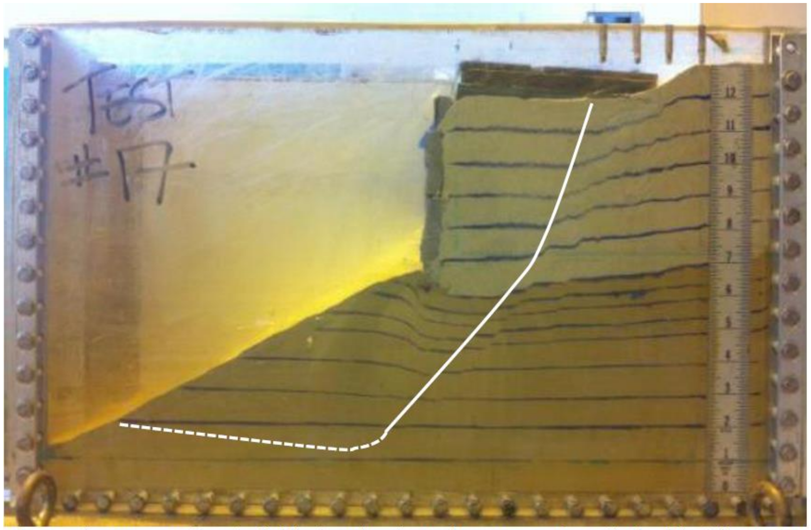


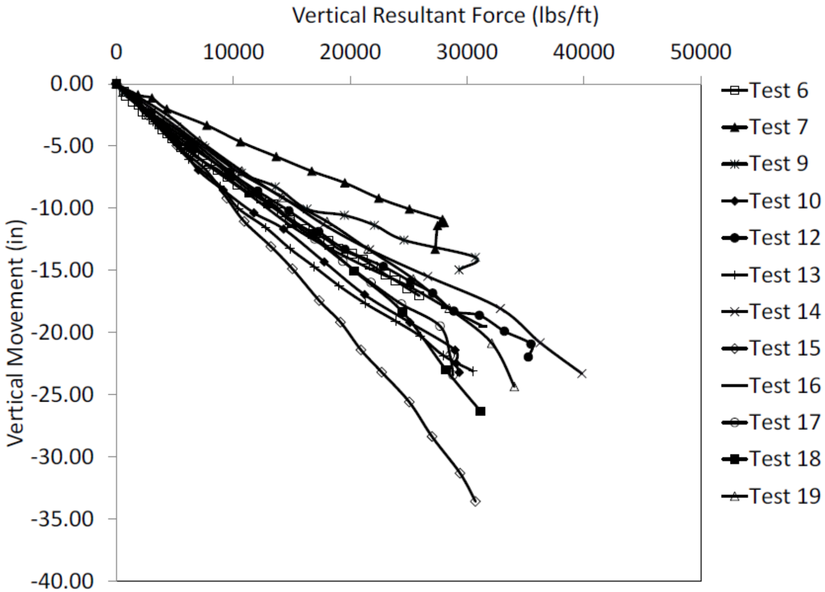
Tests of MSE Walls on Embankments

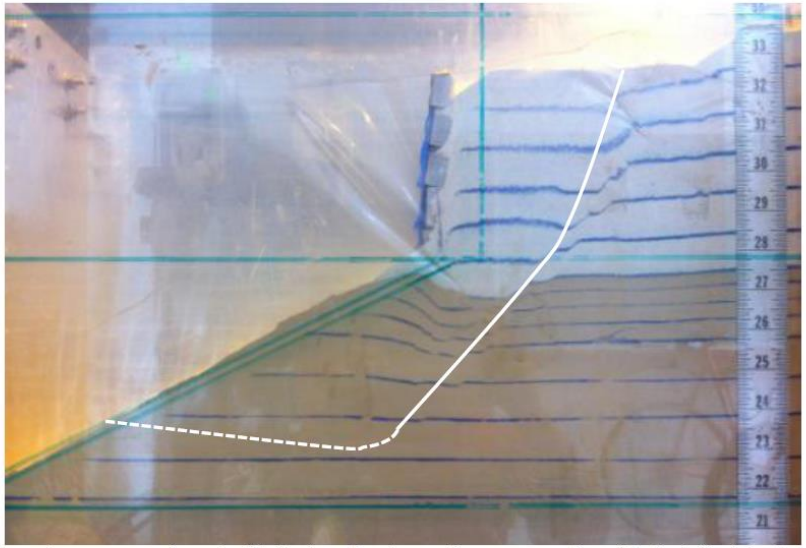
- Models with $\mu_{\phi fs}$ = 26° -27° being evaluated for CV_R
- Previous tests had $\mu_{\phi fs}$ > 32 and did not result in bearing failures and had moisture contents

Test	Backfill				Embankment (Foundation Soil)			
	Unit Weight (pcf) Mean (μ)	Unit Weight CV (%)	Friction Angle (°) Mean (μ)	Friction Angle CV (%)	Unit Weight (pcf) Mean (μ)	Unit Weight CV (%)	Friction Angle (°) Mean (μ)	Friction Angle CV (%)
1	95	7	30	7	100	5	32	5
2	94	8	28	12	105	5	34	5
3	103	6	36	5	105	5	33	5
4	98	1	37	2	107	2	34	2
5	97	1	37	2	107	2	34	2
6	98	2	38	5	86	1	26	1
7	99	13	38	2	90	5	27	4
8	98	1	38	2	86	3	26	2
9	98	1	38	2	85	2	26	2
10	97	1	37	3	82	2	26	2
11	97	1	37	2	90	2	27	2
12	97	1	37	3	82	2	26	2
13	98	1	38	3	85	1	26	1
14	95	1	35	1	83	3	26	2
15	95	1	35	1	83	2	26	2
16	96	1	36	4	84	10	26	3
17	97	2	37	4	82	2	26	2
18	98	4	38	9	82	4	26	4
19	97	2	37	4	85	3	26	2

Observed Load-Displacement and Failures


Observed rupture surface (solid line) and estimated rupture surface (dashed line) in Test 17

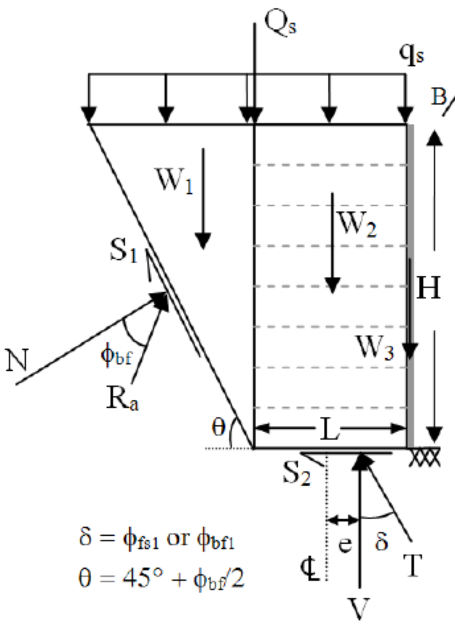


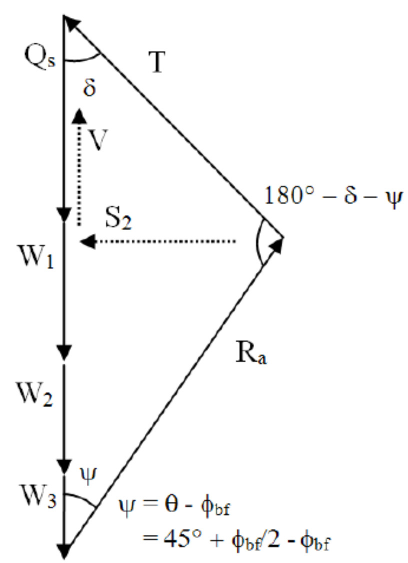

Observed rupture surface (solid line) and estimated rupture surface (dashed line) in Test 10

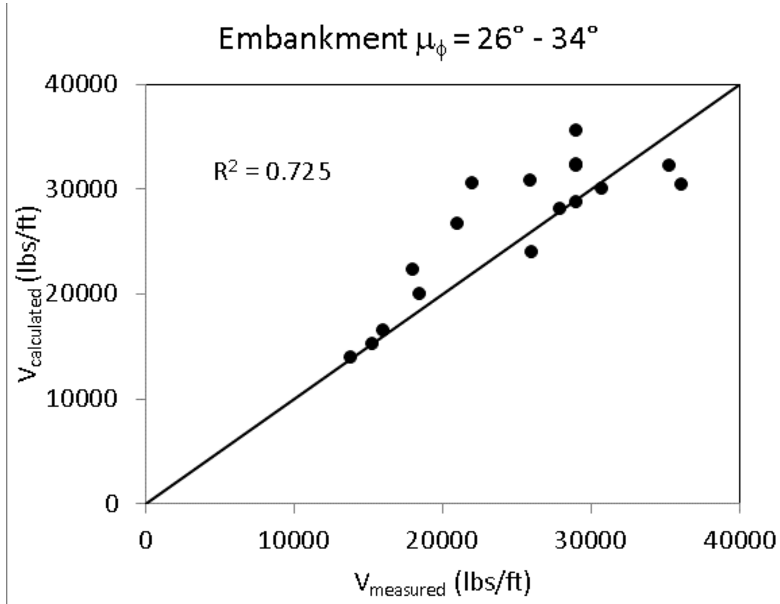

Observed rupture on surface of embankment in Test 10

Validation-Force Equilibrium

- Correlation between $V_{measured}$ and $V_{calculated}$







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

Bearing Capacity Prediction

- Bearing capacity equation

$$q_{u_{pred}} = \frac{1}{2}\gamma L' N'_\gamma i_\gamma$$

- Evaluation with 3 modified soil self weight factors, N_γ' :

- Bowles (1996)

$$N_\gamma' = \frac{N_\gamma}{2} + \frac{N_\gamma}{2}\left[R + \frac{b}{2L}(1-R)\right]$$

where R is ratio of $K_{pmin}/ K_{pmax} = K(-\beta)/K(+\beta)$

$$K_p = \frac{\sin^2(\alpha-\phi)}{\sin^2(\alpha)\sin(\alpha+\phi)\left[1-\sqrt{\frac{\sin(\phi+\phi)\sin(\phi+\beta)}{\sin(\alpha+\phi)\sin(\alpha+\beta)}}\right]^2}$$

- Hansen's

$$g_\gamma = (1-0.5\tan\beta)^5$$

- Vesic's

$$g_\gamma = (1-\tan\beta)^2$$

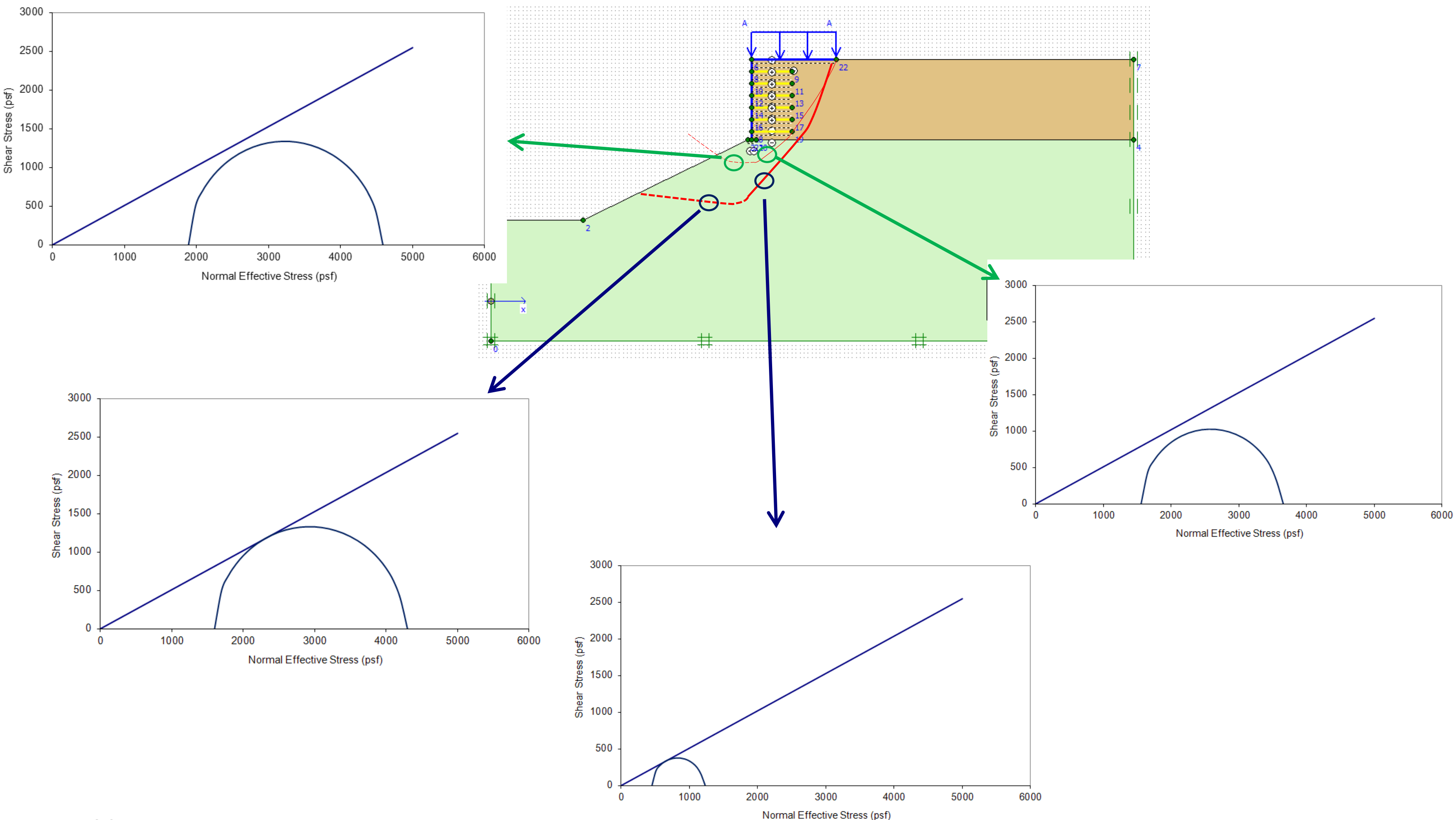
where $N'_\gamma = N_\gamma g_\gamma$

Bearing Capacity Prediction

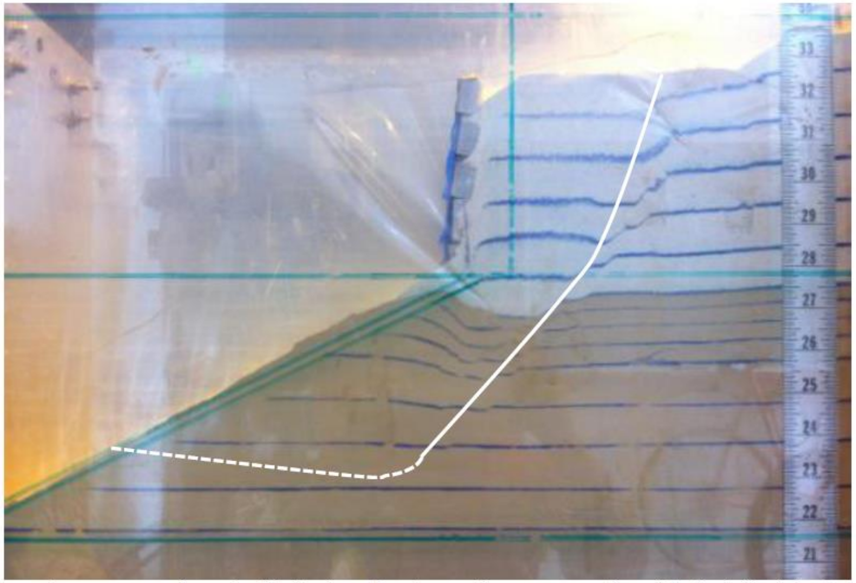
Test	Mean Unit Weight (lbs/ft^2)	Mean Friction Angle (°)	$V_{u\ Hansen}$ (kips/ft)	$V_{u\ Vesic}$ (kips/ft)	$V_{u\ Bowles}$ (kips/ft)	V_{meas} (kips/ft)
1	100	32	2.0	2.3	3.0	13.8
2	105	34	3.6	5.4	7.2	18.5
6	86	26	3.7	2.3	7.5	25.9
7	90	27	4.5	2.8	9.2	27.9
9	86	26	3.5	2.0	7.2	30.7
10	85	26	3.6	2.3	7.4	29.0
11	90	27	3.5	5.8	9.0	29.0
12	82	26	3.5	2.2	7.2	28.5
13	85	26	3.6	2.3	7.4	28.2
14	83	26	3.4	5.7	7.0	32.9
15	83	26	3.4	5.7	7.0	24.5
16	84	26	3.3	5.5	6.8	18.0
17	82	26	3.4	5.7	6.9	28.0
18	82	26	2.9	5.0	5.6	25.0
19	85	26	4.1	6.8	8.3	32.0

- Predictions based on Bowles method use Hansen's N_γ and i_γ, giving the lowest bias (λ) = 3.4
- Vesic's gives bias (λ) = 4.3
- Hansen's gives bias (λ) = 7.2
- If a bearing capacity problem, all methods highly over conservative
- Extents of rupture surfaces suggest failures exhibiting a deeper rupture due to the combined shear on vertical and horizontal plane from slopes and MSE wall

Plaxis Analysis: MSE Walls on Embankments



Observed Rupture Surfaces



Observed rupture surface (solid line) and estimated rupture surface (dashed line) in Test 10

Observed rupture on surface of embankment in Test 10

Conclusions and Recommendations

Sliding Stability

- Resistance factors (Φ)determined for wall heights of 8 ft, 11 ft and 14 ft, L/H = 1, and backfill properties of $\mu_\phi = 32^\circ$ $CV_\phi = 11\%$
- Φ ranged from 0.74 – 0.94 using Rankine’s loading and 0.63 – 0.68 using Coulomb’s loading
- Horizontal load factor (γ_{EH}) was 1.52 using Rankine’s loading and 1.63 using Coulomb’s loading
- AASHTO (2012) recommended design: $\gamma_{EH} = 1.5$, estimating lateral loading using Rankine’s method and $\Phi = 1.0$
- Coulomb’s method leads to conservative Φ’s and are recommended for wall dimensions and soil properties tested

Conclusions and Recommendations

<u>Bearing Stability</u>

- Recommend Vertical load factor (γ_{EV}) = 1.87 be used based on 152 measurements of vertical force. Current practice γ_{EV} = 1.35 (AASHTO, 2012).
- Observed rupture surfaces supported the use of load inclination factors, i_γ
- Recommended $i_\gamma = \left(1 - \frac{S_2}{V}\right)^{1.08}$, 26 < ϕ_{found} < 30 $i_\gamma = \left(1 - \frac{S_2}{V}\right)^{1.55}$, 31 < ϕ_{found} < 33
- Resistance factors (Φ)determined for wall height 20 ft with L/H = 0.5, and foundation soil properties of μ_ϕ = 26° - 30° and 31° - 33° with CV_ϕ = 5%
- Recommend:

  For β = 3.09:

  Φ = 0.47 for $\mu_{\phi \text{foundation soil}}$ = 26° - 30° and Φ = 0.45 for $\mu_{\phi \text{foundation soil}}$ = 31° - 33°

  For β = 2.32:

  Φ = 0.65 for $\mu_{\phi \text{foundation soil}}$ = 26° - 30° and Φ = 0.68 for $\mu_{\phi \text{foundation soil}}$ = 31° - 33°

- AASHTO (2012) recommended Φ = 0.65

Conclusions and Recommendations

MSE Walls on Embankments

- 14 tests exhibited failure – 12 tests with $\mu_{\phi fs} = 26^\circ - 27^\circ$
- Tests with $\mu_{\phi fs} = 26^\circ - 27^\circ$ exhibited deeper rupture surfaces due to the combined shear on vertical and horizontal plane from slopes and MSE wall
- Bearing capacity prediction methods which account for ground inclination, g_γ, (Bowles, Meyerhof, Hansen, and Vesic) are highly over conservative
- Current methods (Bowles, Meyerhof, Hansen, and Vesic) lead to bias, λ, > 3
- Tests suggest bearing capacity of MSE walls on embankments not an issue, passive zone present in bearing capacity failure could not be defined by shape of observed rupture surfaces
- Results indicate the stability was is an overall stability problem (validated with Plaxis model)
- Slope stability analysis should be performed for MSE walls on embankments

Final Report

- McVay, M.C., Bloomquist, D., Wasman, S.J., Drew, G., Lovejoy, A., Pyle, C., O'Brien, R. (2013). "Development of LRFD Resistance Factors for Mechanically Stabilized Earth (MSE) Walls", *FDOT Final Report BDK75 977-22*. (http://www.dot.state.fl.us/research-center/Completed_Proj/Summary_GT/FDOT-BDK75-977-22-rpt.pdf)

Publications

- Wasman, S., McVay, M., Bloomquist, D., Harrison, M., and Lai, P., "*Evaluation of LRFD Resistance Factors and Risk for Mechanically Stabilized Earth Walls*", ASCE Risk Assessment and Management in Geoengineering, Atlanta, Georgia, June 26-28th, 2011.
- Wasman, S.J., McVay, M.C., Bloomquist, D., Lai, P., Jones, L. and Herrera, R., "Determination of LRFD Vertical Load and Resistance Factors for Bearing Capacity of Mechanically Stabilized Earth Walls in Granular Soils", *Geotextiles and Geomembranes*, *(Reviewed with minor revisions).*
- Wasman, S.J., McVay, M.C., Herrera, R., Jones, L., and Lai, P., "Load Factors for Vertical and Horizontal Earth Pressure in LRFD of MSE Walls", *(In Preparation).*
- Wasman, S., Pyle, C., O'Brien, R., and McVay, M.C., "*Centrifuge Modeling of External Stability of Mechanically Stabilized Earth Walls*", Panamerican Conference on Soil Mechanics and Geotechnical Engineering, Buenos Aires, Argentina, November 15th-18th, 2015 *(In Preparation)*.

Thank You!

Questions?